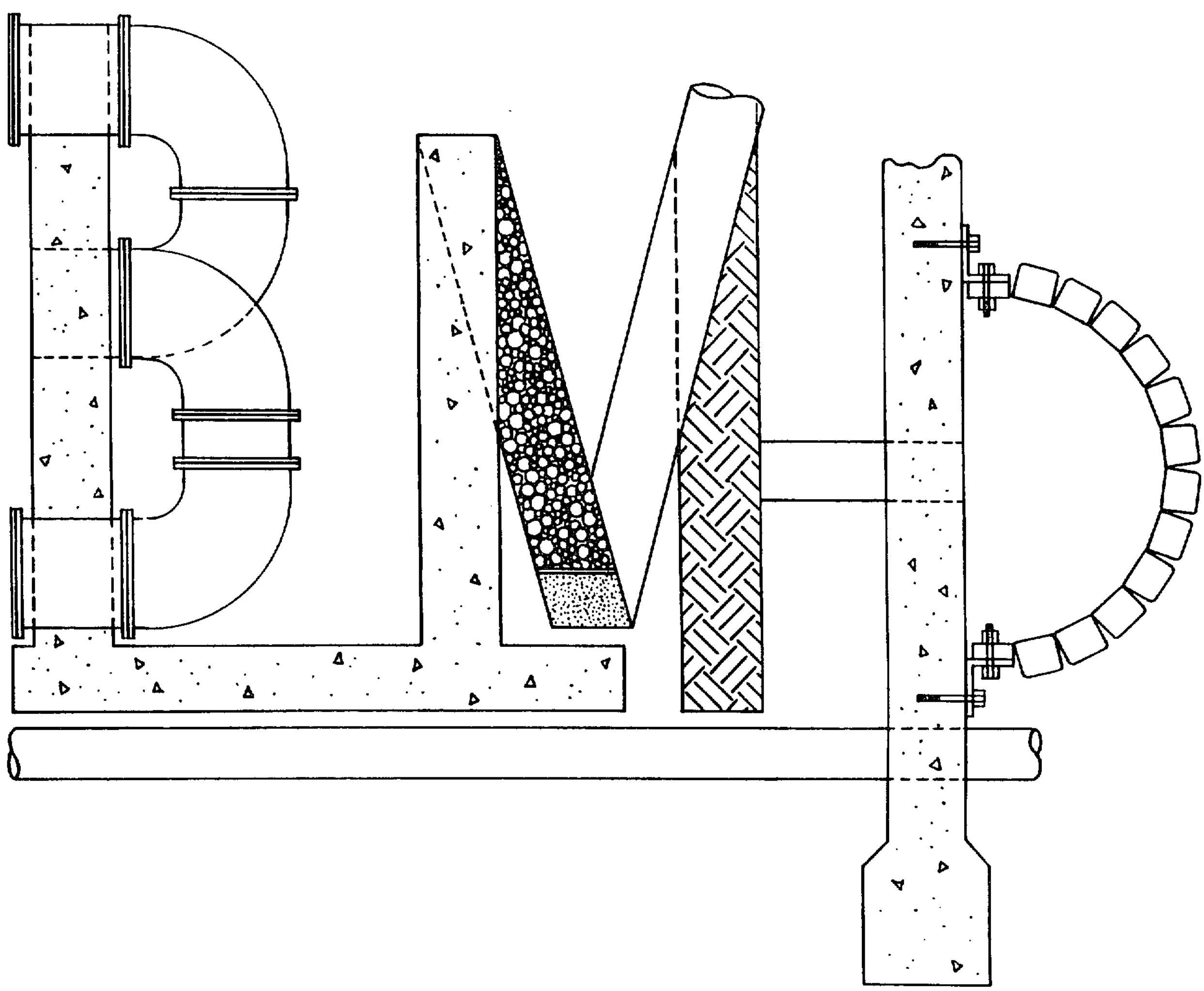
BEST MANAGEMENT PRACTICES DESIGN GUIDANCE MANUAL

FOR HAMPTON ROADS



HAMPTON ROADS PLANNING DISTRICT COMMISSION

DECEMBER 1991

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ARTHUR L. COLLINS, EXECUTIVE DIRECTOR/SECRETARY

February 18, 1992

Chief Administrative Officers
Cities, Counties and Towns
Hampton Roads, Virginia

Re: Regional BMP Design Manual

Enclosed for your use is one (1) copy of the Best Management Practices Design Guidance Manual for Hampton Roads, prepared with consultant assistance by the Hampton Roads Planning District Commission, in cooperation with the staffs of the region's fourteen cities and counties and the Town of Smithfield. Completion of the Manual was made possible by the cooperation and assistance of your staff involved in the Chesapeake Bay and Stormwater Management Programs.

The Manual, which provides guidance on the design of Best Management Practices for stormwater management, is divided into two parts. Part I focuses on a wide variety of practices that are suitable for small sites, especially single lot residential sites. Part II focuses on larger sites and emphasizes detention and retention facilities and the incorporation of wetlands into stormwater management facilities. Both Parts include discussion of maintenance requirements, other operational considerations, facility construction and operational costs and life expectancy.

The Manual is one element of the Commission's regional stormwater management program. It is designed to assist the local governments in complying with the requirements of the Chesapeake Bay Preservation Act, as well as the requirements of the Virginia Stormwater Management Act and the federal Stormwater NPDES Regulations. The Manual is intended to supplement, not replace, existing local government criteria and standards for design, installation and maintenance of stormwater management facilities and Best Management Practices.

The staff of the Hampton Roads Planning District Commission hopes that you and your staffs will find the Best Management Practices Design Guidance Manual for Hampton Roads to be useful in implementing your local stormwater management program. If you have any questions or concerns, please do not hesitate to call.

Sincerely,

Arthur L. Collins

Arthur L. Collins
Executive Director/Secretary

JMC:dfs
Enclosure

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BEST MANAGEMENT PRACTICES

DESIGN GUIDANCE MANUAL

FOR

HAMPTON ROADS VIRGINIA

This report was produced, in part, through financial support from the Chesapeake Bay Local Assistance Department pursuant to Contract No. 91-42 of August 1990 and Unnumbered Contract of June 20, 1990 and from the Virginia Council on the Environment pursuant to Coastal Resources Management Program Grant No. NA90AA- H-CZ796 from the National Oceanic and Atmospheric Administration.

Preparation of this report was included in the HRPDC Work Program for FY 1990-91, approved by the Commission at its Executive Committee Meeting of March 21, 1990 and in the HRPDC Work Program for FY 1991-92, approved by the Commission at its Executive Committee Meeting of March 20, 1991.

Prepared by the Staff of the
Hampton Roads Planning District Commission

December 1991

ACKNOWLEDGEMENTS

From its inception, this project has entailed extensive involvement by staff representatives of the participating local jurisdictions, including the members of the Hampton Roads Chesapeake Bay Committee (Local CBPA Staff Contacts) and supporting staff from other local Departments. Staff from the Chesapeake Bay Local Assistance Department and from the Virginia Council on the Environment also participated heavily in both the technical and administrative aspects of the project. Their participation has been instrumental in the successful completion of this document.

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INTRODUCTION

Previous studies have identified nonpoint source pollution and stormwater management as key water quality issues in many of the estuaries, lakes and rivers of the Hampton Roads area. These issues were addressed in a comprehensive and cooperative fashion by the Hampton Roads Water Quality Agency during the 1970s and early 1980s. As a result of those studies and related programs undertaken throughout the country during the same period, increased attention has been placed on nonpoint source pollution and stormwater management by state and federal regulatory agencies. The cooperative state-EPA Chesapeake Bay Program underscores the importance of these issues insofar as the Chesapeake Bay is concerned. As regulatory programs evolved in the late 1980s, it was determined that a comprehensive and integrated approach to complying with these regulations must be developed.

The Hampton Roads Planning District Commission (HRPDC), through its predecessor the Southeastern Virginia Planning District Commission (SVPDC), addressed these issues through the Regional Stormwater Management Strategy for Southeastern Virginia and the Elizabeth River Basin Environmental Management Program, completed in 1989. Those studies recommended a comprehensive program to be used by the region's local governments to satisfy the requirements of the Chesapeake Bay Preservation Act, the Virginia Stormwater Management Act, the Stormwater Permitting Program of the U.S. Environmental Protection Agency and the State/EPA Nonpoint Source Management Programs. Many of the recommendations were aimed at cooperative approaches to satisfying these requirements. Both studies recommended uniform implementation of Best Management Practices for nonpoint source pollution control.

The Chesapeake Bay Preservation Act (CBPA) was passed by the Virginia General Assembly in 1988. It recognizes the contribution of nonpoint source pollution to the water quality problems of the Chesapeake Bay and its tributaries. The CBPA requires Tidewater localities to designate Preservation Areas, which if improperly developed would lead to water quality degradation, and to incorporate measures into their comprehensive plans and land use controls to protect these areas. The "Chesapeake Bay Preservation Area Designation and Management Regulations" (VR-173-02-01) establish criteria for designating such areas and performance criteria for managing the impacts of development in those areas. This program is mandatory for all localities in Tidewater Virginia. Thus, in Hampton Roads, only the City of Franklin, Southampton County and the six towns in Southampton County are not governed by the program.

Specific performance criteria for stormwater management have been established. They are:

- To prevent a net increase in nonpoint source pollution in runoff from new development;

- To achieve a ten (10) percent reduction in nonpoint source pollution from redevelopment; and,

- To achieve a forty (40) percent reduction in nonpoint source pollution from agricultural and silvicultural uses.

The Best Management Practices Design Guidance Manual for Hampton Roads provides methods for local governments to use in addressing the first two of these criteria. It does not address the criteria that apply to agricultural and silvicultural uses.

The Virginia Stormwater Management Act was enacted by the General Assembly in 1989 and implementing regulations were promulgated by the Department of Conservation and Recreation, Division of Soil and Water Conservation, in 1990. These regulations require that local stormwater management ordinances accomplish the following:

- Require regulated development activities to maintain post-development peak runoff rates at or below pre-development runoff rates;

- Establish minimum technical criteria to control nonpoint source pollution and localized flooding;

- Require the provision of long-term responsibility for and maintenance of stormwater management facilities; and,

- Require local programs to include certain minimum administrative procedures.

The Virginia Stormwater Management Regulations provide for a voluntary program and are only applicable to localities that adopt a stormwater management program. In providing guidance on meeting the stormwater management performance criteria of the CBPA regulations, the Guidance Manual also provides guidance that will assist local governments in satisfying the Stormwater Management Regulations.

The Clean Water Act of 1987 directed the U.S. Environmental Protection Agency to establish a program for permitting municipal and industrial stormwater discharges through the National Pollutant Discharge Elimination System (NPDES) permit program. The Regulations currently apply to stormwater systems in municipalities with populations greater than 100,000, but may apply in the future to smaller municipalities as well. The Regulations require that permit applications be submitted and that management plans be developed as part of that application process. Specific performance standards or discharge limits have not yet been established. The Guidance Manual should facilitate local efforts to comply with such standards when they are promulgated.

As the region's localities have devised specific approaches to implementing these various stormwater and nonpoint source management requirements, the need for comparable and consistent approaches to facility design has been underscored. The development community appears to agree, at least in concept, with this idea.

Based on this well-documented need, in 1990 the HRPDC, in cooperation with the participating localities, undertook a project to develop a Regional Design Manual for Best Management Practices (BMPs). This Manual was to document the most appropriate and effective BMPs for use in the Hampton Roads region. The selected BMPs were to be sufficient to enable development in the region to comply with the requirements of the Chesapeake Bay Preservation Act, as well as with other state and federal stormwater management regulations. Completion of the project was facilitated by financial assistance from the Chesapeake Bay Local Assistance Department and the Virginia Council on the Environment. Engineering consultant assistance was also obtained.

The following objectives were established for this project. They are:

- To develop a uniform, regional approach to implementation of state and federal stormwater and nonpoint source management programs.
- To develop BMPs which satisfy the CBPA stormwater management performance criteria.
- To determine the most cost-effective, preferred BMPS for application in Southeastern Virginia.
- To develop standardized engineering design standards and specifications for BMPs to be used in Southeastern Virginia.

As the Manual evolved over the last two years, the focus shifted to providing common guidance for use by both local planning and engineering staffs and the development community in designing and developing stormwater management facilities.

The Best Management Practices Design Guidance Manual for Hampton Roads has been completed in two Phases. The Guidance Manual is not intended to supercede the public facility and stormwater management design requirements established by local governments. It is intended to supplement those requirements and provide additional guidance for use in designing stormwater facilities that comply with evolving state and federal stormwater management requirements. Users are referred to the local public facility and stormwater management facility design manuals for locality-specific design requirements. In addition, users are referred to standard engineering texts and handbooks for detailed design information on hydraulics and hydrology.

Both phases of the Guidance Manual address general planning considerations, including an overview of the Chesapeake Bay Preservation Act and implementing regulations, the Virginia Stormwater Management Act and its implementing regulations and the EPA Stormwater NPDES Permit Regulations. For each Best Management Practice addressed, design guidance is provided, as is information about construction and maintenance requirements and costs and life expectancy.

Phase I of the Guidance Manual focuses on on-site BMPs and, in particular, on BMPs which are suitable for single family dwelling units and small commercial sites. It includes detailed design criteria and specifications, as well as examples, on each of the practices. Practices included in this phase include Biofiltration, Grassed Swales, and Filter Strips. They also include Dry Wells, Infiltration Trenches and Basins, Underground Storage Trenches, Porous and Modular Pavements, Grit-Oil Separators and Water Quality Inlets. Suggestions on combining one or more of these practices in certain situations are also provided.

Phase II encompasses regional BMPs. It addresses detention and retention facilities, the incorporation of wetlands features into such facilities and the retrofitting of existing stormwater management facilities so that they provide additional water quality benefit. Phase II of the Guidance Manual stresses the value of multi-objective stormwater management planning and facility design. Detailed checklists are provided for construction, inspection and maintenance of such facilities. Because these facilities are typically large-scale, subject to a wide variety of site-specific and watershed-specific constraints, and require design flexibility and innovation, detailed design examples are not included. Detailed guidance on the establishment of wetlands in stormwater management facilities is included.

The Guidance Manual is structured to facilitate updating as new research is completed and as additional experience with BMP installation and operation in the Hampton Roads area is gained. Each chapter on an individual practice is structured as a stand- alone section that can be reproduced and provided to property owners and designers. This organization also facilitates replacement of current chapters with new information as it becomes available.

Preparation of the Best Management Practices Design Guidance Manual for Hampton Roads is one element of a comprehensive regional program in stormwater and environmental management. Other elements of this program include:

1. Regional Stormwater Management Strategy for Southeastern Virginia, SVPDC staff, 1989. This study documented the evolving state of stormwater management regulation at the state and federal level. It

outlines a technical and institutional strategy that can be used by the region's local governments to comply with these regulations. While specifically applicable to the eight jurisdictions that, at that time, were part of the Southeastern Virginia Planning District Commission, it is generally applicable to all of the localities in the Hampton Roads Planning District.

2. Stormwater Management Financing Strategy for Hampton Roads Virginia, HRPDC staff, 1991. This study documented the need for additional authority for local governments to use in financing stormwater management programs to meet state and federal regulations. It recommends use of stormwater utilities as an equitable means of accomplishing that.

3. Model Environmental Assessment Procedure, HRPDC staff, 1992. This study outlines one approach to evaluating the environmental impacts of development proposals. It also provides guidance on conducting the water quality impact assessments required under the Chesapeake Bay Preservation Act.

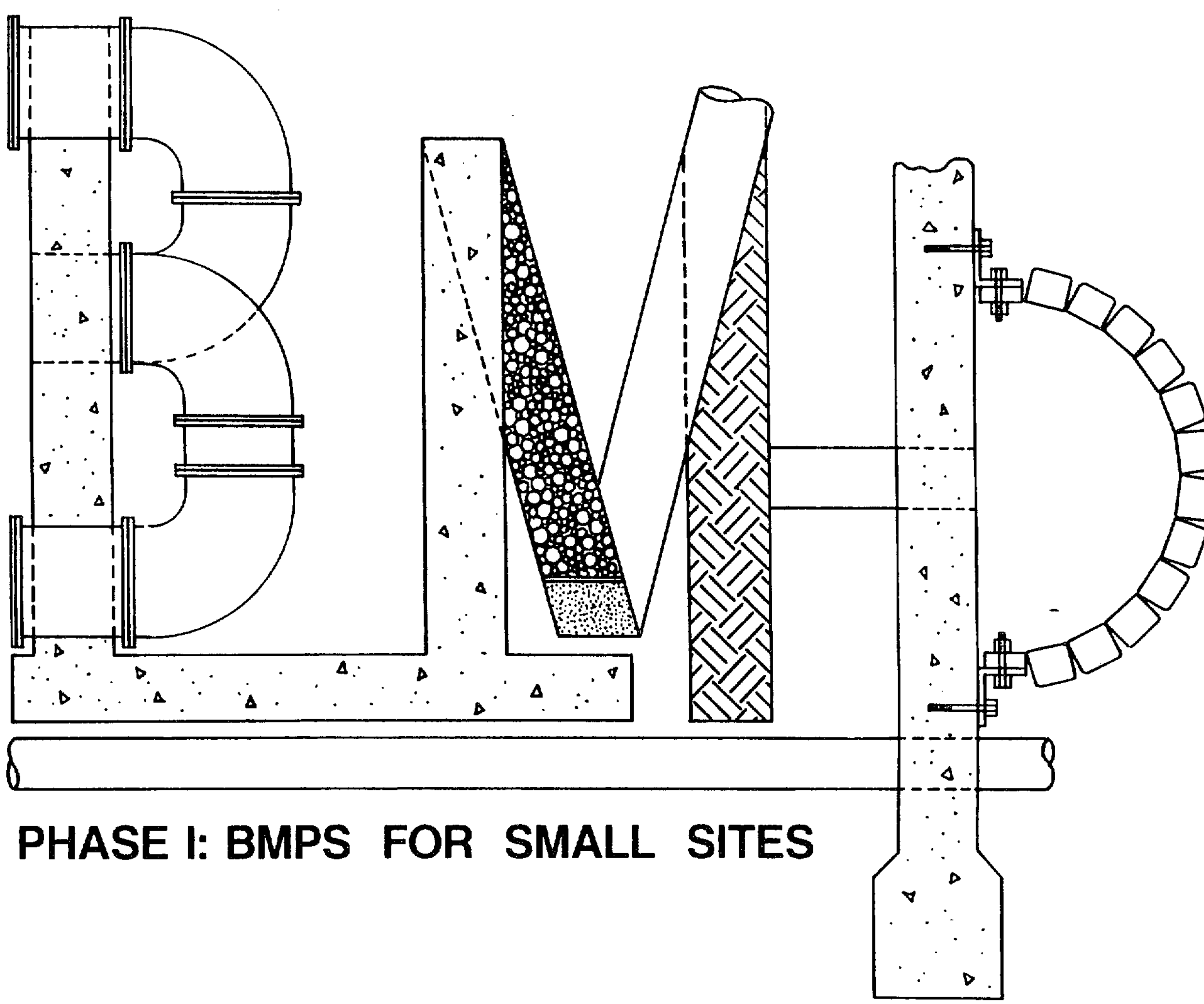
4. Vegetative Practices Design Guidance is being prepared by the HRPDC staff in cooperation with staff from the region's localities. This study will constitute Phase III of the BMP Manual and will be incorporated into the Manual when it is completed. It reflects a recognition on the part of local government staffs that structural BMPs may not be the most cost-effective approach to stormwater management for small, residential sites in the Hampton Roads region.

5. BMP Tracking System is being developed by the HRPDC staff in cooperation with staff members from the region's localities. This project will provide a computerized system for monitoring BMP installation and maintenance to assist localities in ensuring that property owners comply with the CBPA requirements for BMP maintenance.

6. Regional Stormwater Management Coordination Process. Through this activity, the staff of the HRPDC is facilitating regular meetings of the local government staff involved in stormwater management programs. These meetings provide an opportunity for exchange of ideas and experience. In a related activity, the Hampton Roads Municipal Communicators, in cooperation with the HRPDC and local government staffs, has developed educational materials, including brochures and a video, on stormwater management.

BEST MANAGEMENT PRACTICES DESIGN GUIDANCE MANUAL

FOR HAMPTON ROADS



PHASE I: BMPS FOR SMALL SITES

HAMPTON ROADS PLANNING DISTRICT COMMISSION

DECEMBER 1991

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PHASE I: BMPS FOR SMALL SITES

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Prepared by SDN Water Resources
in cooperation with the Staff of the
Hampton Roads Planning District Commission

December 1990

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The Hampton Roads Planning District Commission and the Chesapeake Bay Local Assistance Department request the cooperation of recipients of this Manual in complying with these restrictions.

BMP DESIGN GUIDANCE MANUAL

JANUARY, 1991

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Chesapeake, Virginia

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APPENDICES

1. INTRODUCTION

1.1 Purpose and Scope

The Hampton Roads Planning District Commission (HRPDC), on behalf of its member local governments, has undertaken development of a cooperative regional stormwater management program. A variety of regional efforts in this regard have been underway for several years to assist the localities to respond to various state and federal programs as well as local needs. They have ranged from studies of stormwater quantity issues in the early 1970s to studies of stormwater quality and nonpoint source management issues in the late 1970s and 1980s.

In 1988, the Virginia General Assembly enacted the Chesapeake Bay Preservation Act (CBPA), which established requirements for local land use planning and regulation to protect water quality. These requirements include fairly stringent performance criteria for stormwater quality management on a development specific basis. The CBPA and its requirements affect development activities in several Southeastern Virginia localities. They include the County of Isle of Wight, the Cities of Chesapeake, Norfolk, Portsmouth, Suffolk, and Virginia Beach, and the Towns of Smithfield and Windsor.

To facilitate local compliance with the stormwater management requirements of the CBPA, the localities requested the HRPDC to undertake development of a Best Management Practices (BMPs) Design Guidance Manual for Southeastern Virginia. SDN Water Resources was contracted in August 1990 to provide assistance in the development of this Manual. The following objectives were established for preparing the Design Guidance Manual:

- To develop BMPs which satisfy the CBPA stormwater management performance criteria.
- To determine the most cost-effective, preferred BMPs for application in Southeastern Virginia.

- To develop standardized engineering standards and specifications for BMPs to be used in Southeastern Virginia.

As the project evolved, the Manual has become a design guidance document. It provides guidance to the localities on those BMPs which are most appropriate for use in Southeastern Virginia. While the primary purpose is to assist the localities in complying with the CBPA and its implementing regulations, the Manual should also assist in addressing the quality aspects of the Virginia Stormwater Management Act and the National Pollution Discharge Elimination System stormwater permitting requirements.

The Manual is not intended to be a comprehensive stormwater management Manual, insofar as hydrology and hydraulics are concerned. Users should consult standard engineering texts and handbooks for detailed information on these subjects.

SDN Water Resources has prepared planning guidelines, design criteria, and standard details for the following BMPs. A brief description of each BMP in the Design Guidance Manual follows. Section numbers are keyed to the chapter numbers for each BMP. Detailed discussion of each BMP including design criteria, maintenance requirements, costs, and construction specifications can be found in subsequent chapters.

1.2 General Planning Considerations

To select a BMP for a site, factors to be considered are the infiltration rate of the soils of the site and the ground water table. The total contributing area to the BMP is also a factor. Other physical factors to be considered for selection of a BMP for a site are proximity to water supply wells and foundations, slope of the site, and specific use of the site.

1.3 Biofiltration

Biofiltration as a BMP utilizes the interaction of soil and vegetative cover to remove pollutants from surface runoff. It functions like a swale or filter strip.

1.4 Grassed Swale with Check Dam

Grassed swales are typically used in low density areas as an alternative to curb and gutter drainage systems. The pollutants are filtered out by the grass and subsoil. Check dams may be used to temporarily pond runoff, allowing infiltration over a period of time. They cannot, however, accommodate major runoff events and may lead to other downstream BMPs.

1.5 Filter Strip

Also known as buffer zones, filter strips are similar to grassed swales except that they are wider. They should be at least 20 feet wide and not be used on slopes greater than 15%. Filter strips are usually vegetated and accept evenly distributed sheet flow. There are secondary benefits including aesthetics, wildlife habitat, and noise screening.

1.6 Dry Well

A dry well utilizes the concept of infiltration to remove pollutants from surface runoff from rooftops. The dry well is a variation of the infiltration trench and is designed exclusively for runoff from rooftops. Roof leaders are extended to a stone filled trench located a minimum of ten (10) feet from the building foundation.

1.7 Infiltration Trench

An infiltration trench is typically three (3) to eight (8) feet deep and filled with stone to create an underground reservoir. Runoff can either drain from the reservoir into the underlying soil (exfiltration) or be collected by underdrains and directed to an outflow. Typically, infiltration trenches can only accommodate limited quantities of runoff and are used for sites of less than ten (10) acres in size.

1.8 Infiltration Basin

Whereas infiltration trenches serve small sites, infiltration basins can serve drainage areas up to 50 acres. They are designed to promote exfiltration through the underlying material. They should be vegetated and often include devices which prevent coarse sediment from entering the basin as well as emergency spillways for extreme storm events.

1.9 Underground Storage Trench

An underground storage trench is designed to remove sediments and hydrocarbons from parking lots and commercial sites where there is not enough space for infiltration systems.

1.10 Porous Pavement

Porous pavement detains and minimizes the effects of runoff containing traffic generated pollutants. Both soluble and very fine grained pollutants are removed by infiltration through the stone reservoir and into the underlying soil. This BMP has a number of shortcomings which generally confine it to low volume traffic areas such as parking lots. It consists of a graded aggregate cemented with asphalt cement, with numerous voids to provide a high rate of permeability.

1.11 Grid/Modular Pavement

Using the same concept as porous pavement, this type of pervious pavement consists of a grid made of concrete, clay bricks, or granite sets. The void areas of the grid are filled with a pervious material such as sod, gravel, or sand.

1.12 Grit-Oil Separator

A grit-oil separator is used to remove oil and grit deposits from runoff from parking lot areas and commercial sites.

1.13 Water Quality Inlet

Water quality inlets are typically used to serve parking lots one (1) acre or less in size, and are primarily used in combination with other BMPs as a pretreatment facility to remove coarse sediment particles.

1.14 Regional BMPs

Phase II of the BMP Design Guidance Manual will incorporate regional BMPs like Detention and Retention Ponds, Extended Detention/Retention Ponds, and Detention/Retention Ponds with wetland bottoms.

2. GENERAL PLANNING CONSIDERATIONS

The planning of BMPs requires collection of information about underlying soils and the groundwater table. All BMPs which utilize infiltration are dependent on the ability of the underlying soil to infiltrate storm runoff and not be inundated by groundwater. The following paragraphs provide guidelines for selecting suitable BMPs for a specific site.

2.1 Soil Information

A critical element in selecting a BMP for a specific site is collecting and analyzing the soil information. An initial indication of the soil can be made from existing Soil Survey Maps prepared by the U.S. Soil Conservation Service. A detailed soil survey for each site where an infiltration BMP is to be located should be conducted. This is normally performed by taking samples from a drilled hole. The hole should be drilled at least four (4) feet below the anticipated design depth of the BMP. The collected samples should be graded in the laboratory and the infiltration rate determined. The minimum infiltration rate is the rate at which the water passes through the soil profile during saturated conditions. It is measured in inches per hour. The hydrologic soil properties are obtained by identifying the soil textures (gradation test). Table 1 lists soil texture classes and their typical infiltration rates.

Table 1 - Hydrologic Soil Properties		
Texture Class	Minimum Infiltration Rate (Inches/Hour)	Hydrologic Soil Group
Sand	8.27	A
Loamy Sand	2.41	A
Sandy Loam	1.02	B
Loam	0.52	B
Silt Loam	0.27	C
Sandy Clay Loam	0.17	C
Clay Loam	0.09	D
Silty Clay Loam	0.06	D
Sandy Clay	0.05	D
Silty Clay	0.04	D
Clay	0.02	D

Source: "Controlling Urban Runoff" - Metropolitan Washington Council of Governments.
Note: The CBPA regulations define soil with an infiltration rate greater than six (6) inches per hour as highly permeable.

Soil textures with a minimum infiltration rate greater than or equal to 0.27 inches per hour are generally suitable for infiltration practices. Soil textures with a minimum infiltration rate close to, but less than 0.27 inches per hour may be used for infiltration practices with careful analysis of the soil profile.

Hydrologic Soil Group classification indicates the minimum rate of infiltration obtained for bare soils after prolonged wetting. These groups are classified as A, B, C, and D. Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. Group B soils have moderate infiltration rates when thoroughly wetted. Group C soils have low infiltration rates when thoroughly wetted. Group D soils have high runoff potential and have very low infiltration rates when thoroughly wetted.

2.2 Groundwater Information

Another critical element in selecting a BMP for a specific site is the location of the seasonal high groundwater table. This can be determined by observing static water elevation in borings. The groundwater elevations should be determined after a period of eight (8) to 12 hours and not after the boring is taken. Generalized information about the groundwater table may also be obtained from local health departments and the SCS.

Groundwater information is important to determine the safe distance between the bottom of the BMP structure and the seasonal high groundwater table. Infiltration BMPs should be located in areas where the bottom of the structure is two (2) to four (4) feet above the seasonal high groundwater table. This distance should also protect against the flooding of the structure due to the rise of the water table. A flooded infiltration BMP will be ineffective. The Virginia Stormwater Management Regulations require that the invert of the infiltration BMPs should be four (4) feet above the seasonal high groundwater table.

2.3 Other Site Selection Criteria

Selection of a BMP for a specific site depends on the contributing area of that BMP. Infiltration practices such as infiltration trenches, porous pavement, grid/modular pavement, and underground storage trenches are practical and economical for contributing areas up to five (5) acres. Dry wells are suitable for rooftop areas up to one (1) acre. Infiltration basins may serve areas up to 50 acres. Grassed swales with check dams are suitable for areas up to 30 acres. Biofiltration can be used for areas up to ten (10) acres. Filter strips are suitable for areas up to five (5) acres.

Topographic conditions also determine the feasibility of a BMP for a specific site. These site conditions include slopes, proximity of water supply wells, and building foundations. The use of infiltration BMPs on fill material is not recommended due to the possibility of slope failures when the fill material is saturated. Table 2 presents a matrix that shows the site selection criteria for all BMPs. A solid dot indicates that a BMP is feasible. Open space in the matrix indicates a restriction. Other site selection restrictions for each BMP are also indicated.

Maximum feasible depth of a BMP is a function of the minimum infiltration rate of the soil. The permeability of the soil underlying a BMP and the void ratio of the stone aggregate reservoir dictates the maximum feasible depths of infiltration BMPs.

TABLE-2 BMPs SELECTION CRITERIA

BEST MANAGEMENT PRACTICES (BMPs)	AREA SERVED (ACRES								SOIL TYPE AND MINIMUM INFILTRATION RATE (INCHES/HR.)											OTHER RESTRICTIONS						
	0-5	5-10	10-15	15-20	20-25	25-30	30-50	50+	SAND	LOAMY SAND	SANDY LOAM	LOAM	SILT LOAM	SANDY CLAY LOAM	CLAY LOAM	SILTY CLAY LOAM	SANDY CLAY	SILTY CLAY	CLAY	GROUND WATER TABLE (ft.)	SLOPE (%)	PROXIMITY TO WELLS (ft.)	PROXIMITY TO BUILDINGS (ft.)	BUFFER REQUIREMENTS (ft.)	SITE CONSTRAINT	NORMAL DEPTH RANGE (ft.)
									A	A	B	B	C	C	D	D	D	D	D							
									8.27	2.41	1.02	0.52	0.27	0.17	0.09	0.06	0.05	0.04	0.02							
BIOFILTRATION	●	●									●	●	●	●	●	●				1-2	< 4					
DRY WELL	●								●	●	●	●								2-4	<20	>100	> 10	> 20	RES-IDENTIAL ROOF TOP	2-6
INFILTRATION TRENCH	●	●							●	●	●	●								2-4	<20	>100	> 10	> 20		2-6
INFILTRATION BASIN		●	●	●	●	●	●		●	●	●	●								2-4	<20	>100	> 10	> 20		2-6
GRASSED SWALES (WITH CHECK DAMS)	●	●	●	●	●	●			●	●	●	●	●	●						1-2	< 5		> 10			1/2-2
FILTER STRIPS *	●								●	●	●	●	●	●						1-2	<20					
POROUS PAVEMENT	●								●	●	●	●								2-4	< 5			> 20	NO HEAVY TRAFFIC	1-4
UNDERGROUND STORAGE	●								●	●	●	●								2-4		>100	> 10	> 20		
GRID/MODULAR PAVEMENT	●								●	●	●	●								2-4	< 5				ONLY IN PARKING AREAS	
GRIT-OIL SEPARATOR	●								●	●	●	●	●	●	●	●	●	●	●						ONLY IN COM-MERCIAL AREA	
WATER QUALITY INLET	●								●	●	●	●	●	●	●	●	●	●	●						ONLY IN COM-MERCIAL AREA	
DETENTION PONDS			●	●	●	●	●	●	●	●	●	●	●	●	●	●										
RETENTION PONDS			●	●	●	●	●	●				●	●	●	●	●	●	●	●							
EXTENDED DETENTION/ RETENTION PONDS			●	●	●	●	●	●	●	●	●	●	●	●	●	●	●									
DETENTION/RETENTION WITH WETLAND BOTTOMS			●	●	●	●	●	●	●	●	●	●	●	●	●	●										

* NOTE: A FILTER STRIP, DESIGNED AS A BMP, DIFFERS FROM THE BUFFER AREA REQUIRED BY THE CBPA REGULATIONS. REFER TO SECTION 5.

2.4 Pollutant Removal Efficiency of BMPs

Pollutants exist in particulate or soluble forms, or as a mix of both forms. Particulate pollutants, such as sediment and lead, are removed by settling and filtering. Soluble pollutants, such as nitrate, phosphate, and trace metals, are removed through biological uptake by bacteria, algae, rooted aquatic plants, or vegetation.

Total phosphorus has been selected by the Chesapeake Bay Local Assistance Department (CBLAD) as the keystone pollutant to be removed. By removing total phosphorus from the urban runoff, other urban pollutants are also removed.

Table 3 lists an estimated removal efficiency of each BMP. Each removal efficiency has been selected based on review of current literature and is intended as a general guideline only. More precise numbers for removal efficiency will be forthcoming in the future as a result of more monitoring and evaluation of BMPs.

Table 3 - BMP Removal Efficiency	
BMP	REMOVAL EFFICIENCY, PERCENT
BIOFILTRATION	40 - 80
DRY WELL	50 - 70
INFILTRATION TRENCH	50 - 70
INFILTRATION BASIN	50 - 70
GRASS SWALES (W/CHECK DAMS)	10 - 20
FILTER STRIPS	20 - 50
POROUS PAVEMENT	50 - 70
UNDERGROUND STORAGE	50 - 70
GRID/MODULAR PAVEMENT	50 - 70
GRIT-OIL SEPARATOR	10 - 25
WATER QUALITY INLET	10 -25
DETENTION PONDS	20 - 50
RETENTION PONDS	35 - 65
EXTENDED DETENTION/RETENTION PONDS	25 - 60
DETENTION/RETENTION PONDS WITH WETLAND BOTTOMS	40 - 75
Source: Smith Demer Normann, 1990 Note: Removal efficiencies refer to Total Phosphorus.	

2.5 Design Storm

The CBPA has established the following technical criteria:

> *For new development, the post-development nonpoint source pollution runoff load cannot exceed the pre-development load based on average land cover conditions. This is referred to as a "no net increase" standard.*
>
> *For redevelopment sites not served by BMPs, the post-development nonpoint source pollution runoff load must be 90 percent or less than the pre-development load for that site. This is referred to as a "10 percent reduction" standard.*

A 100-foot vegetative buffer is required landward of specified sensitive shoreline features. Under certain circumstances, the width of the buffer area can be reduced if equivalent water quality benefits are achieved by installing BMPs on site.

Pollution runoff loads are computed for an average annual rainfall depth of 45 inches for Tidewater.

Virginia Stormwater Management Regulations established the following technical criteria:

> *A stormwater management plan for a land development project shall be developed so that from the site, the post-development peak runoff rate from a two-year (2 yr.) storm and a ten-year (10 yr.) storm, considered individually, shall not exceed their respective pre-development rates.*
>
> *For infiltration facilities, the water quality volume must be completely infiltrated within 48 hours. Water quality volume means the volume equal to the first 0.5 inch of runoff multiplied by the total area of the land development project.*

All BMP's can be designed for a specific storm or for the first flush runoff volume. The storage volume of all BMPs can also be sized based on:

(1) The runoff produced by a one-inch (1") storm over the contributing site area.

(2) 0.5 inch of runoff per impervious acre in the contributing site area (first flush).

(3) The runoff per impervious acre produced by a one-inch (1") storm.

(4) 0.5 inch of runoff in the contributing site area (Virginia Stormwater Management Regulations).

3. BIOFILTRATION

3.1 Description

Biofiltration is a BMP which utilizes the concept of treating stormwater runoff by direct contact with the soil and vegetation. Biofiltration involves storm runoff being transported over a vegetative surface and is similar to a filter strip or a swale. The storm runoff can also be ponded in an area containing emergent wetland plants. Pollutants such as sediments and trace elements in the runoff are removed by biological uptake and infiltration through the soil. By providing sufficient residence time for the storm runoff, pollutant removal and infiltration can be accomplished. A schematic of biofiltration is shown on Figure 1.

3.2 Applicability

Biofiltration in its many forms is used as a BMP for removing pollutants from storm runoff. It can be used in residential areas and adjacent to highways.

3.3 Design Criteria

3.3.1 Soil Permeability

Biofiltration can be used with soils having infiltration rates ranging from 1.02 inches per hour to 0.06 inches per hour. These infiltration rates are associated with soil textural groups of sandy loam, silt-loam, sandy clay loam, clay loam, and silty clay loam.

3.3.2 Length

This BMP should have a hydraulic length of at least 200 feet. If length is less, width needs to be made larger to provide the equivalent residence times. Sufficient residence time is needed [illegible] sediments to be removed through the mechanism of settling.

3.3.3 Side Slopes

Side slopes should be as flat as feasible with three to one (3:1) being the recommended value.

3.3.4 Longitudinal Slope

Longitudinal slopes should be in the range of two (2) to four (4) percent. Slopes less than two (2) percent can be used with underdrains to avoid persistent pooling of runoff. The longitudinal slopes of two (2) to four (4) percent are impossible to attain in a large portion of Southeastern Virginia.

3.3.5 Vegetation Cover

Vegetation cover as outlined in chapters 4 and 5 should be used for biofilters. Vegetation selected should be suitable for the site. The type of vegetation selected depends on the type of underlying soil. Maintenance requirements should also be considered in selecting the type of vegetation.

3.3.6 Shape

A parabolic shape for biofilters is preferred. Initially, the swale channel can be constructed as a trapezoid. With time, trapezoidal shapes tend to become parabolic due to the growth of vegetation and settlement of solids.

3.3.7 Depth of Flow

The design depth of flow should be at least two (2) inches less than the winter vegetation height.

3.3.8 Groundwater Table

The seasonal high groundwater table should be between one (1) and two (2) feet below the ground surface. A groundwater table high enough to provide moisture to the vegetation during the dry season but not high enough to create long periods of saturation is ideal. In areas with high saturation, emergent wetland plants can be planted.

3.3.9 Velocity

Based on the slope parameters of the biofilter selected, the velocity of storm runoff should not exceed 1.5 feet per second. This velocity will assist in the removal of suspended solids in the storm runoff. If the velocity exceeds 1.5 feet per second, suspended solids in the storm runoff cannot be easily settled.

3.3.10 Manning's Value of Vegetation

The following Manning's n values are recommended:

Table 4 - Manning's Value of Vegetation

Height of Vegetation	n Value
Dense grass up to 6 inches tall	0.07
Dense grass 6-12 inches tall	0.10
Dense grass > 12 inches tall	0.20
Wetland Plants	0.07

Source: Biofiltration Systems for Storm Runoff, Water Quality Control - Richard R. Horner, December 1988.
Note: Refer also to Chapter 5 in the Virginia Erosion and Sediment Control Handbook.

3.3.11 Depth of Flow

Biofiltration can be designed for a specific storm or the first flush runoff volume. The storage volume can be sized based on 0.5 inches of runoff per impervious acre in the contributing site area (first flush).

Biofiltration is normally designed as a water quality trench. As such, a significant portion of the runoff volume will bypass the trench and is not infiltrated. An earthen berm protected by stone should be installed at the end of the biofilter. Height of the berm should be the design depth of flow and freeboard. The berm shall facilitate ponding of the storm runoff. Stone protection of the berm should prevent erosion from overflow. Provision for overflow can also be made by providing a notch in the berm.

3.4 Design Examples

Design a biofilter swale for a discharge of two (2) CFS. This discharge is computed to be generated from a commercial site of one (1) acre. The vegetative cover selected is dense grass with a Manning's n value of 0.07. The swale is to be designed for a depth of four (4) inches. Longitudinal slope of the Biofilter swale should not exceed two (2) percent. Design the swale for lengths of 200 feet and 150 feet.

DESIGN OF BIOFILTRATION SWALE

Project: 3.4 BIOFILTER SWALE DESIGN FOR LENGTH < 200 FT. (150 FT.)

----->>> Input Parameters <<<-----

Discharge Rate Q (cfs): 2.0

Manning's Coefficient of Vegetation Cover: 0.07

Depth of Flow(In): 4.0

Longitudinal Slope (%): 2.0

----->>> Output Values <<<-----

Top Width (Ft): 11

Velocity(Ft/Sec): 0.8

DESIGN OF BIOFILTRATION SWALE

Project: 3.4 BIOFILTER SWALE DESIGN FOR 200 FT.

----->>> Input Parameters <<<-----

Discharge Rate Q (cfs): 2.0

Manning's Coefficient of Vegetation Cover: 0.07

Depth of Flow(In): 4.0

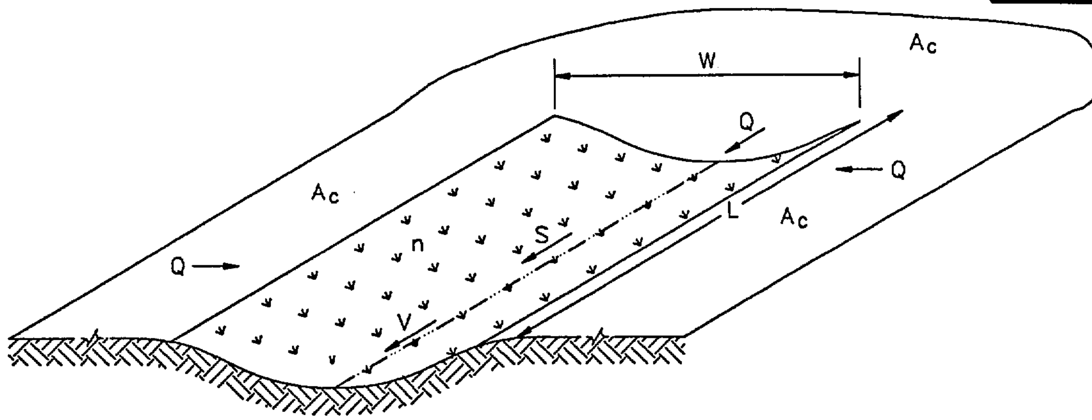
Longitudinal Slope (%): 2.0

----->>> Output Values <<<-----

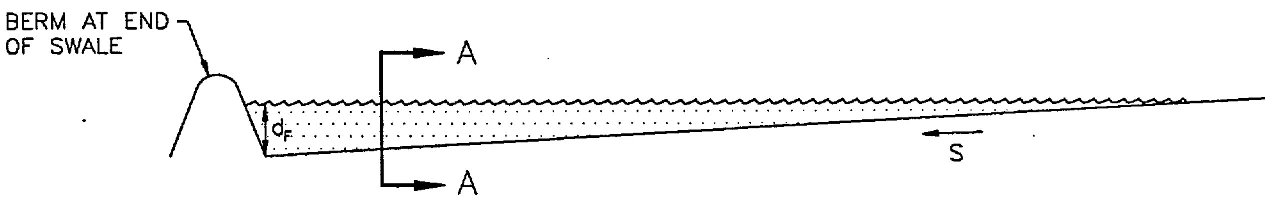
Top Width (Ft): 8

Velocity(Ft/Sec): 1.1

FIGURE 1



SCHEMATIC SECTION A–A



SIDE VIEW

Q = Discharge Rate Q (cfs)

n = Manning's Coefficient of Vegetation Cover

d_F = Depth of Flow (In)

S = Longitudinal Slope (%)

L = Length of Swale (Ft)

W = Design Width of Swale (Ft)

V = Design Velocity of Swale (Ft/Sec)

A = Design Area of Swale (Sq Ft)

T = Resident Time (Sec)

For Length of Swale = 200 Ft:

$$W = \frac{Q\,n}{0.76\left(\frac{d_F}{12}\right)^{\frac{5}{3}}\left(\frac{S}{100}\right)^{\frac{1}{2}}}$$

$$A = \frac{2}{3}\,W\,\frac{d_F}{12}$$

$$V = \frac{Q}{A}$$

For Length of Swale other than 200 Ft calculate for length 200 ft and also do :

$$T = \frac{200\ \text{Ft}}{V} \qquad V = \frac{Q}{A}$$

$$A = \frac{T\,Q}{L} \qquad W = \frac{1.5\,A}{\frac{d_F}{12}}$$

BIOFILTRATION SCHEMATIC

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HAMPTON ROADS
PLANNING DISTRICT COMMISSION

REVS'D:	DRAWN: K.F.F.
CHK'D:	CHK'D: V.P.M
DATE:	DATE: OCT. 1990
	SCALE: N.T.S.
REVS'D:	
CHK'D:	
DATE:	PAGE 20

3.5 Maintenance Requirements

Maintenance requirements of biofilters are minimal, except for mowing and removal of sediments. Once the vegetation has been established, mowing should be performed on an as needed basis. It should be performed at least once a year and the vegetation clippings should not be allowed to decay in the biofiltration facility. Sediments should be removed whenever the volume of the facility is determined to be inadequate and ponding occurs. Continuous ponding may require mosquito control.

Proper inspection should be performed during the construction of the biofiltration facility. It should be inspected regularly during the period when vegetation is being established. After the vegetation has been established, once a year inspection should be enough.

Maintenance costs of biofilters depend on various factors such as size of the BMP, type of vegetation, frequency of mowing, and can vary from facility to facility.

3.6 Life Expectancy

Biofiltration, if properly constructed, inspected, and maintained, can have a life expectancy of several years. With regular maintenance, it can function properly for 20 years. After more examples of this BMP have been constructed and monitored a more exact number for life expectancy can be established.

3.7 Cost

Biofilter establishment costs are similar to swale or filter strip costs. Costs for planting emergent wetlands could range from $1,000-$3,000/acre for material plus labor.

3.8 Construction Specifications

3.8.1 Site Preparation

Install needed erosion and sediment control practices such as silt fences, dikes, contour ripping, erosion stops, channel lines, sediment traps, and sediment basins.

3.8.2 Vegetation

Select vegetation according to the site conditions. Use guidelines and specifications outlined in section 5.8. Select fine, close-growing, water-resistent grasses.

3.8.3 Slope

The minimum slope for biofilters should normally be two (2) percent and the maximum four (4) percent. A flatter slope can be specified if it is known that the ponded runoff will drain and will not be subject to persistent water pooling.

3.8.4 Compaction

Avoid compaction during construction. If compaction occurs, till before planting vegetation to restore soil infiltration capacity.

4. GRASSED SWALE WITH CHECK DAM

4.1 Description

Concentrated storm runoff can be impounded behind check dams constructed of railroad ties or stone berms to induce infiltration. As the storm runoff overtops the check dams, it can be directed to flow over vegetated drainage swales with gentle slopes. The gentle slopes with vegetative cover provide non-erosive flow velocities. The combination of low velocities and vegetative cover provide an opportunity for sediments to settle out. Grassed swale with check dam schematic is shown on Figure 2. Typical grassed swale details are shown on Figures 3 and 4.

4.2 Applicability

Grassed swales with check dams are mostly applicable in residential developments of low to moderate density where the impervious cover is relatively small. Swales are usually located in a drainage easement at the side or back of residential lots or in highway medians. Swales should not be designed for large, infrequent storms.

Swales with check dams can be used in combination with infiltration trenches. The trench should be constructed under the swale. The pool created by the check dam increases the volume of surface runoff infiltrating into the trench. The grassed swale helps to impede the transport of suspended solids downstream. Swales are not generally capable of removing soluble pollutants, such as nutrients, because of insufficient residence time and vegetation types.

This BMP is an alternate to curb and gutter sections and is less expensive.

4.3 Design Criteria

4.3.1 Soil Permeability

The permeability or final infiltration rate of the underlying soil should be equal to or greater than 0.17 inches per hour. Those soil textural classes that have slow infiltration rates should not be considered for grassed swales. Thus, the suitable textural classes of the soil underlying the swale are sand, loamy sand, sandy loam, loam, and sandy clay loam.

4.3.2 Swale Gradient

Grassed swales with check dams should not be constructed with bottom slopes of greater than five (5) percent. Minimum slopes can be as close to zero (0) as drainage will permit.

4.3.3 Groundwater Table

The seasonal high groundwater table should be at least one (1) to two (2) feet below the bottom of the grassed swale.

4.3.4 Design Storm

Grassed swales with check dams can be designed for a specific storm or for the first flush runoff volume. The storm runoff volume to b[illegible]ed can be based on 0.5 inch of runoff per impervious acre in the contributing site area (first flush).

Runoff associated with less frequent large storms will bypass the grassed swales without being treated and overtop the check dams.

4.3.5 Storage Time/Maximum Draining Time

It is recommended in the literature that the maximum allowable ponding time in swales be 24 hours.

4.3.6 Permissible Velocity

If a large design storm is used to design the swale, the velocity of flow expected from the design storm should not exceed the permissible velocity for the type of vegetative lining used for the swale. Table 5 lists the permissible velocities for various covers.

Table 5 - Permissible Velocities for Various Ground Covers				
No.	Cover	Slope Range Percent (%)	Permissible Velocity (feet/second)	
			Erosion Resistant Soils	Easily Eroded Soils
1	Bermudagrass (Bynodon Dactylon)	0-5	8	6
2	Kentucky 31 Tall Fescue (Festuca Arundinacea)	0-5	7	5
3	Grass-legume mixture	0-5	4	3
4	Red Fescue Redtop (Agrostis Alba) Lespedeza Servicea Alfalfa	0-5	3.5	2.5
5	Annuals * Common Lespedeza Sudan Grass Small Grain Ryegrass	0-5	3	2
6	Rock Riprap Section (for temporary construction)	5-10	8	6.5

* Annuals are used on mild slopes (less than three (3) percent) or as temporary protection until permanent covers are established. Use on slopes steeper than five (5) percent is not recommended.

Source: Maryland Standards and Specifications for Soil Erosion and Sediment Control, 1983.

4.3.7 Capacity

The swale must have sufficient capacity to pass the peak discharge rate of the design storm. The grassed channel should be designed in accordance with the Manning formula.

$$Q = \frac{1.49}{n} A R^{2/3} \sqrt{S}$$

where Q = peak flow rate of the design storm, in cubic feet per second (cfs)

n = Manning's roughness coefficient

A = Cross-sectional area of the swale (ft.^{2})

R = Hydraulic Radius (ft.)

S = Longitudinal slope (ft./ft.)

4.3.8 Side Slope

The side slopes of the vegetated swale should not exceed three to one (3:1) and for swales lined with riprap two to one (2:1).

4.3.9 Cross Section

Swale channel cross-sections can be trapezoid, parabolic, or V-shaped. The trapezoidal swale shape is the preferred section due to its ease of construction. With time, trapezoidal shapes tend to become parabolic due to the growth of vegetation and settlement of solids.

4.4 Design Examples

A one-acre (1 ac.) lot is to be developed for constructing a house with a roof-top area of 2,000 square feet. The soil borings on the site indicate that the soil is silt loam with an infiltration rate of 0.27 inches per hour. The depth of the seasonal high water table is determined to be five (5) feet deep.

Design a swale to treat the runoff. Swale is assumed to be constructed adjacent to the width of the one-acre lot assumed to be 500 feet. The side slopes of the swale should be three to one (3:1) and the bottom width of the check dams are assumed to be ten (10) feet. The longitudinal slope of the swale is two (2) percent.

DESIGN OF GRASS SWALE WITH CHECK DAMS

Project: 4.4 DESIGN EXAMPLE FOR 500 FT. SWALE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.27

Maximum Allowable Ponding Time(Hrs): 24

Depth to Seasonal High Groundwater Table(Ft): 5.00

Min. Dist. from Swale Bottom to Groundwater Table(Ft): 2.00

***** Maximum Swale Depth(Ft): 0.54 *****

----->>> Design Input Parameters <<<-----

Depth of Swale (Ft): 0.54

Bottom Width of Check Dam (Ft): 10.0

Side Slope Ratio (N:1): 3.0

Longitudinal Slope (%): 2.0

Total Hydraulic Length of Swale(Ft): 500

----->>> Design Output Values <<<-----

Length of Swale Behind Check Dam(Ft): 26

Number of Check Dams Required: 19

Top Width of Swale(Ft): 13.24

Design a swale for a row of houses in a residential development. Runoff from this residential development is computed to generate 0.4 inches of runoff from a one-inch (1") storm. The contributing site area is two (2) acres. The soil characteristics at the site suggest an infiltration rate of 1.2 inches per hour. Groundwater information reveals the seasonal depth of water to be three (3) feet. Side slopes for the swale to be designed are three to one (3:1) and the bottom width of check dams is 10 feet. The longitudinal slope of the swale is three (3) percent.

DESIGN OF GRASS SWALE WITH CHECK DAMS

(UNKNOWN LENGTH)

Project: 4.4 GRASS SWALE DESIGN FOR UNKNOWN LENGTH ----->>>

Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 1.20

Maximum Allowable Ponding Time(Hrs): 24

Depth to Seasonal High Groundwater Table(Ft): 3.00

Min. Dist. from Swale Bottom to Groundwater Table(Ft): 2.00

******* Maximum Swale Depth(Ft): 1.0 *******

----->>> Design Input Parameters <<<-----

Depth of Swale (Ft): 1.00

Bottom Width of Check Dam (Ft): 10.00

Side Slope Ratio (N:1): 3.0

Longitudinal Slope (%): 3.0

----->>> Calculated Total Hydraulic Swale Length <<<-----

Increase in Runoff Depth(In): 0.40

Contributing Site Area(Sq Ft): 87120

Swale Filling Time(Hrs): 2

Amount of Rainfall(In): 1.00

Total Hydraulic Length of Swale(Ft): 347

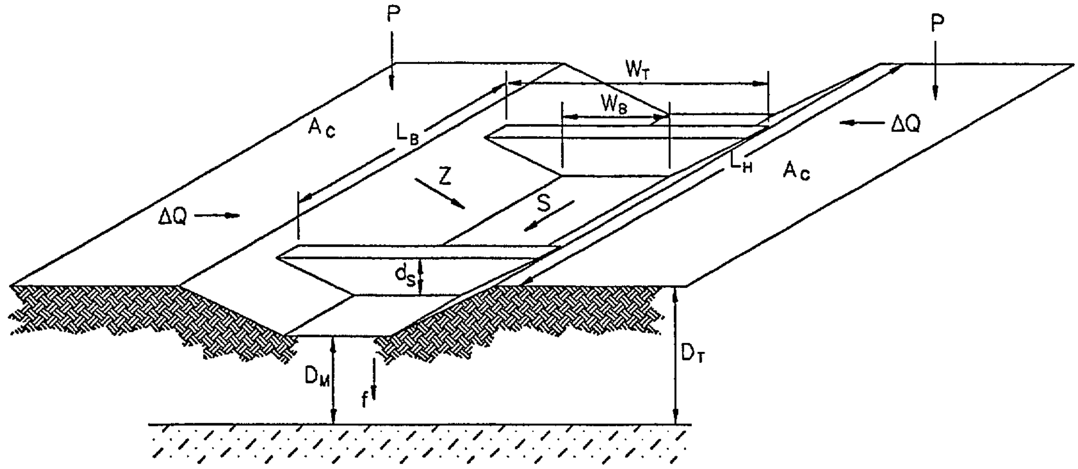
----->>> Design Output Values <<<-----

Length of Swale Behind Check Dam(Ft): 35

Number of Check Dams Required: 10

Top Width of Swale (Ft): 16.00

FIGURE 2



f = Infiltration Rate(In/Hr)

T_P = Maximum Allowable Ponding Time(Hrs)

D_T = Depth to Seasonal High Groundwater Table(Ft)

D_M = Min Dist from Swale bottom to Groundwater Table(Ft)

d_S = Depth of Swale(Ft)

W_B = Bottom Width of Check Dam (Ft)

Z = Side Slope Ratio (N:1)

S = Longitudinal Slope (%)

L_B = Length Behind Check Dams (Ft)

L_H = Total Hydraulic Length of Swale (Ft)

ΔQ = Increase in Runoff Depth(In)

A_C = Contributing Drainage Area(Sq Ft)

T = Swale Filling Time(Hrs)

P = Amount of Rainfall(In)

W_T = Top Width of Check Dam (Ft) = $W_B + 2\,d_S\,Z$

To Compute L_H if not known:

$$L_H = \frac{\frac{\Delta Q}{12} A_C}{\frac{d_S}{4}(W_T + W_B) + W_T\left(\frac{f}{12}T - \frac{P}{12}\right)}$$

$$\text{Volume of Storage (Cu Ft)} = \frac{d_S (W_T + W_B) L_H}{4}$$

$$L_B = \frac{d_S}{\frac{S}{100}} \qquad \text{\# Check Dams} = \frac{L_H}{L_B}$$

$$\text{Adjusted } L_B = \frac{L_H}{\text{\# Check Dams}}$$

GRASSED SWALE W/ CHK DAMS SCHEMATIC

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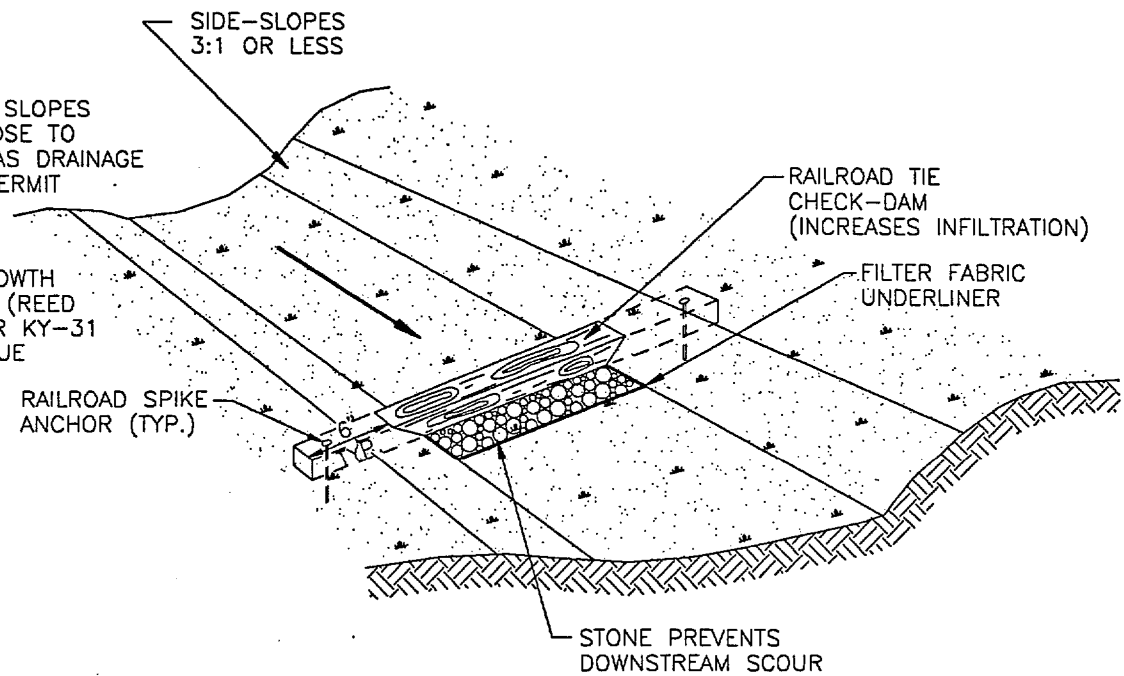
FIGURE 3



GRASSED SWALE W/ CHK DAMS DETAIL

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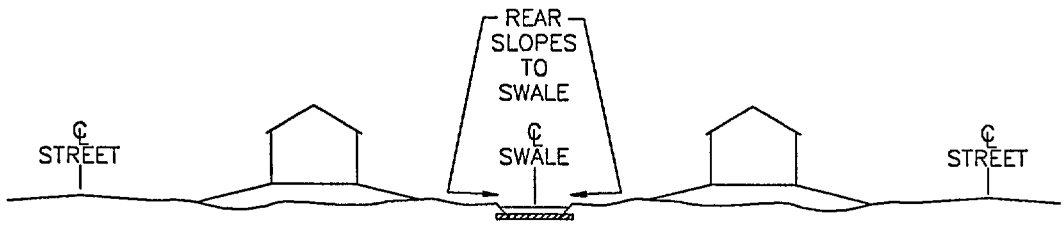
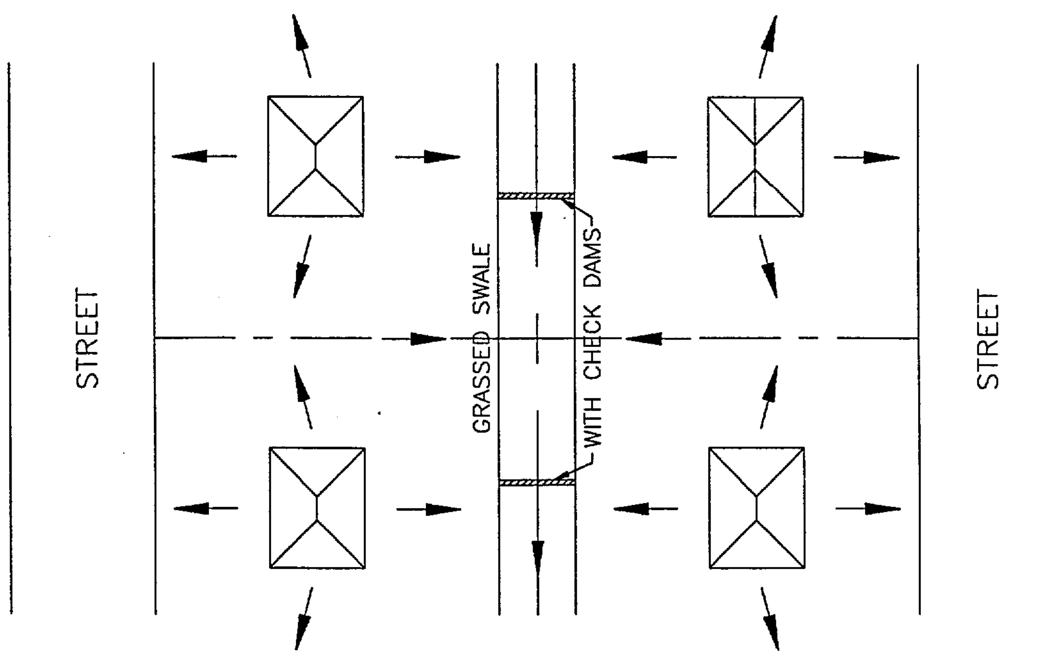
FIGURE 4



SIDE VIEW



TOP VIEW

GRASSED SWALE W/ CHK DAMS

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4.5 Maintenance Requirements

Swale maintenance is very minor if the swale is properly installed. An annual inspection of the site is recommended to assure that the swale is functioning properly. Swale maintenance consists mainly of keeping the vegetative cover dense and vigorous and involves periodic mowing, and reseeding of bare spots. Mowing of the swale too close to the ground (scalping) should be avoided. Vegetative cover should remain at least three (3) inches. Any sediments collected behind the check dam should be periodically removed. Some temporary nuisance problems like mosquito breeding may develop and can be alleviated by periodic mowing. Persistent ponding in swales can be eliminated by debris cleaning, mowing, and drilling holes in the bottom of the swale.

Maintenance costs for swales depend on various factors such as size of the swale, type of vegetation, number of check dams, frequency of mowing, and can vary for each swale.

4.6 Life Expectancy

Grassed swales have long been used as ditches for highways. Their life expectancy as water quality facilities has not been monitored sufficiently. If properly constructed, inspected, and maintained, it is estimated that they can last for 20 years.

4.7 Cost

The typical costs for establishing the vegetative cover by various seeding methods are given below:

Table 6 - Typical Costs for Establishing Vegetative Cover	
Grading	\$3 - \$8 per cubic yard
Hydroseeding (with mulch and fertilizer	\$ 1,500 - \$1,750 per acre
Conventional Seeding	\$1,200 - \$1,600 per acre
Seed/straw mulching cost for a swale	\$2 - \$4 per linear foot
Railroad Ties 6" x 8"	\$4 - \$6 per linear foot

4.8 Construction Specifications

Check dams can be constructed of railroad ties, wood logs, gabions or other suitable material. Wood logs should be pressure treated logs or made of water resistant tree species such as cedar, hemlock, swamp oak or locust. Earthen check dams are not recommended as they can be easily eroded. Check dams should be installed perpendicular to the direction of flow and can be anchored into the sloping sides of the channel. The toe of the check dam should be protected by riprap which should be placed over a suitable geotextile fabric.

Gabions used as check dams should be made of hexagonal triple twist mesh with PVC coated galvanized steel wire. The maximum linear dimension of the mesh opening shall not exceed 4.5 inches and the area of the mesh opening shall not exceed ten (10) square inches.

Stone or riprap for gabions should be sized according to the following criteria:

Table 7 - Gabion Sizing Criteria

Basket Thickness (Inches)	Stone Size (Inches)
6	3 - 5
9	4 - 7
12	4 - 7
18	4 - 7
36	4 - 12

The stone or riprap shall consist of field stone or rough unhewn quarry stone. The stone shall be hard and angular and of a quality that will not disintegrate on exposure to water or weathering. The specific gravity of the individual stones shall be at least 2.5.

A small notch or depression shall be provided in the gabion dam to create a flow channel in the center of the dam for overflows.

5. FILTER STRIP

5.1 Description

A vegetative filter strip is an area of vegetative cover through which the storm runoff flows before it leaves the site. The storm runoff must be evenly distributed across the filter strip and the flow velocity of the runoff should be reduced. Concentrated flow from the site across the filter strip should be avoided. Concentrated flows tend to form a channel. Once a channel is formed, filter strips will not perform as designed. The flow can be evenly distributed across the filter strip by using level spreaders. A vegetated filter strip detail with spreader is shown on Figure 7. A vegetated filter strip can provide the following benefits.

- Serves as an effective method of reducing sediment yield by protecting the soil from rainfall impact energy.
- Reduces runoff by reducing overland flow velocities, increasing the time of concentration, and increasing infiltration.
- Removes suspended sediment in overland flow by filtering, absorption, and gravity sedimentation as the flow velocity is reduced.

5.2 Applicability

Vegetative filter strips can be used as the sole BMP or in combination with other BMPs. All BMP structures should be surrounded by vegetative filter strips to alleviate the sediment load being delivered to the BMP.

5.3 Design Criteria

5.3.1 Flow

The vegetative filter should be used to control overland sheet flow only. If the filter will be subject to any concentrated flows, such as found at low points in parking lots or grass areas, then a level spreader should be used to establish sheet flow.

5.3.2 Selecting the Type of Vegetation

The selection of vegetative materials ranges from using existing vegetation to specifying a vegetation mix tailored to suit the characteristics of the site. Table 9 should be used as a guide in selecting the vegetation type.

5.3.3 Slope Characteristics

The effectiveness of vegetative filters as sediment control devices decreases with increasing slope. Filter strips are not effective on slopes greater than 15 percent.

5.3.4 Runoff

When filter strips are used in treating sediment-laden runoff, the following shall be considered:

(1) Good drainage to ensure satisfactory performance.

(2) A level spreader at the inlet to ensure uniform distribution of flow.

(3) An adequate filter area and length of flow to provide the desired treatment.

(4) Slopes less than five (5) percent are more effective; steeper slopes require a greater area and length of flow to achieve the same effectiveness.

(5) Provisions for mowing and removing undesirable vegetation to maintain the effectiveness of the filter area.

5.3.5 Length of Filter Strip

The minimum length of filter strip used in conjunction with all other BMPs should be 20 feet.

Additional guidelines to assist the designer in calculating the trap efficiency of an existing vegetative buffer strip, or the length of vegetative filter required to provide a specific trap efficiency are provided below.

5.3.6 Graphical Solution

A solution for computing the sediment trap efficiency of a vegetative filter strip can be represented graphically. Figure 5 shows the relationship between trap efficiency (T_R) and the length and slope of the filter strip, as well as the roughness coefficient of the vegetation (Manning's n). The required length of a filter strip is very sensitive to variation in the trap efficiency as it approaches 100 percent, indicating that a small incremental increase in the trap efficiency requires a considerable addition in the filter strip. The curves also suggest that a significant trap efficiency (up to 75 percent) may be achieved at relatively short filter strip lengths. Figure 5 assumes a coarse silt material.

The trap efficiency for other soil textures may also be determined using Figure 5. The settling velocity of sediment particles manifests the appropriate trap efficiencies that are attainable using filter strips for a particular particle size. In general, the greater the settling velocity, the higher the trap efficiency per length of filter strip. For example, the ratio of the settling velocities for a coarse silt and a fine silt is 4.9. Thus, the filter strip length obtained from Figure 5 should be multiplied by this ratio to obtain the filter strip length for a fine silt. This would provide the same trap efficiency indicated on Figure 5. The settling velocity ratio of coarse silt to medium silt, fine sands, and medium sands are 1.3, 0.02, and 0.005 respectively. These ratios are shown on Table 8.

Table 8 - Effective Buffer Strip Length

Type of Soil	Ratio of Settling Velocity	Effective Buffer Strip Length
Coarse Silt	1	1 x length from Figure 5
Fine Silt	4.9	4.9 x length from Figure 5
Medium Silt	1.3	1.3 x length from Figure 5
Fine Sands	0.02	0.02 x length from Figure 5
Medium Sands	0.005	0.005 x length from Figure 5

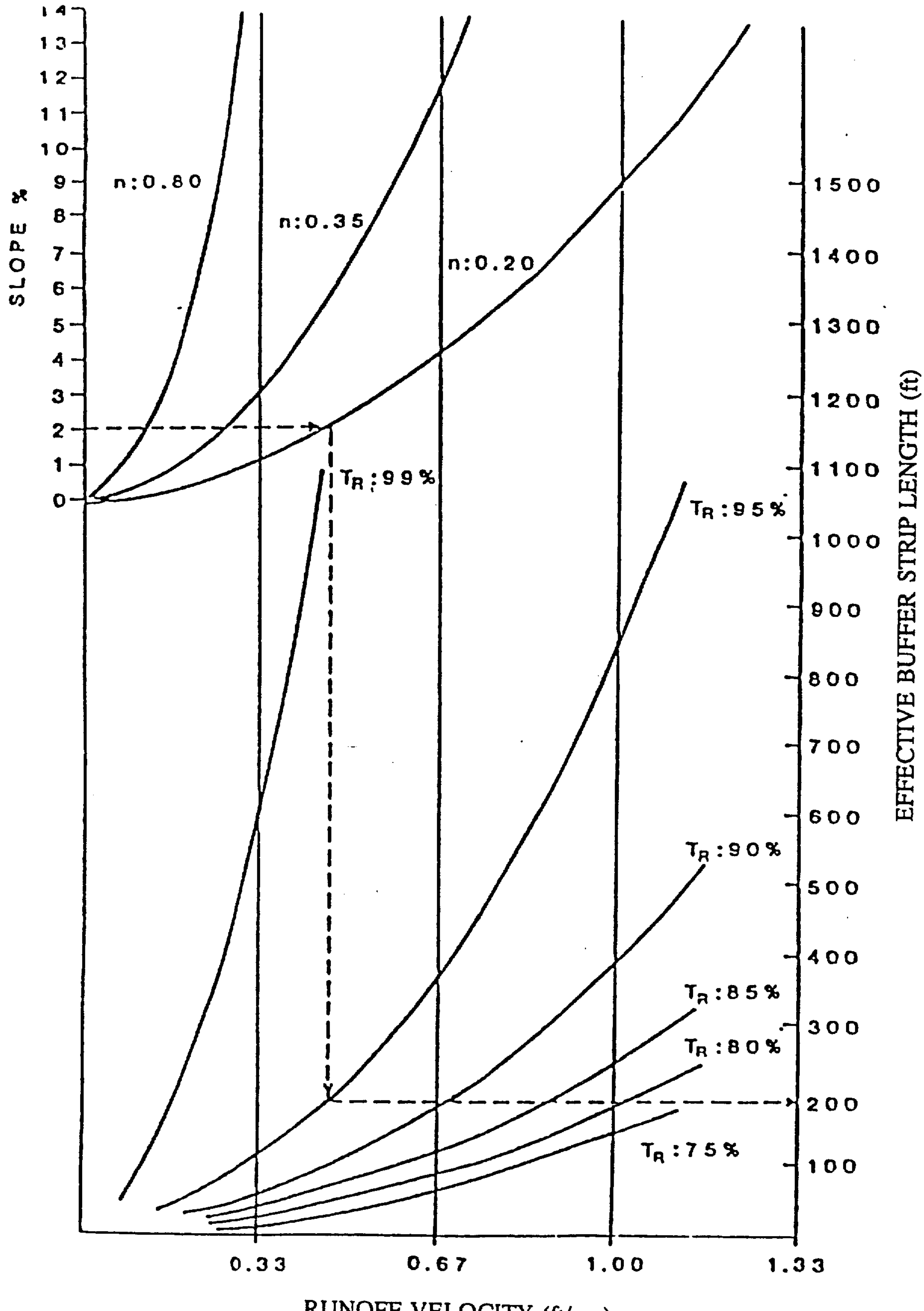


Figure 5 Effective Buffer Length Determination for Trap Efficiencies (T_R) of 75 to 99 percent (Buffer length for coarse silt.)

Source: Wong and McCuen, 1982

5.4 Design Examples

Example 1

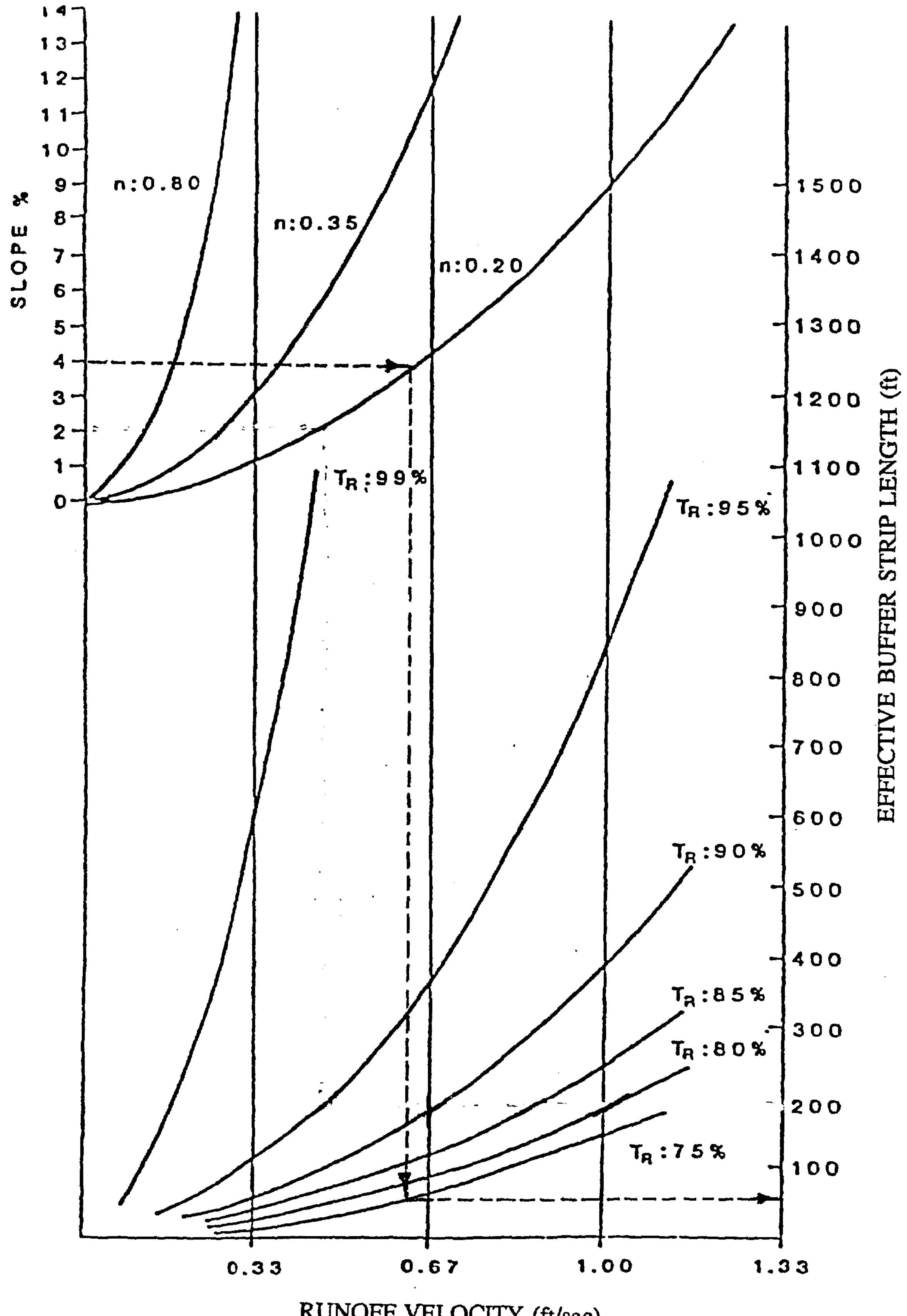
Design the length of a vegetated filter strip required to remove 95% of sediments from the runoff. The type of vegetation selected for filter strip is dense grass of a height greater than 12 inches. Slope of the filter strip will be two (2) percent. The type of soil for the filter strip is coarse silt. Use Figure 5 to determine the effective length of the filter strip.

From Table 4, select the Manning's value (n) for dense grass greater than 12 inches. For a slope of two (2) percent, draw a straight line to intersect the curve for selected n value of 0.20. Draw a vertical line from the point of intersection to intersect 95 percent trap efficiency (T_R) curve. Draw a straight line from this point to find the effective filter strip length of 200 feet.

Example 2

Design a vegetative filter strip for underlying soil of medium silt. The slope of the filter strip is four (4) percent and the type of vegetation selected is dense grass with a Manning's roughness coefficient (n) of 0.20. The filter strip is assumed to trap 75% of sediments from the runoff. Use Figure 6 to determine the effective length of the filter strip.

By using the procedure outlined in example 1, effective filter strip length is approximately 55 feet. As the underlying soil is medium silt, multiply this length by 1.3 (from Table 8) to arrive at effective filter strip length of 72 feet.

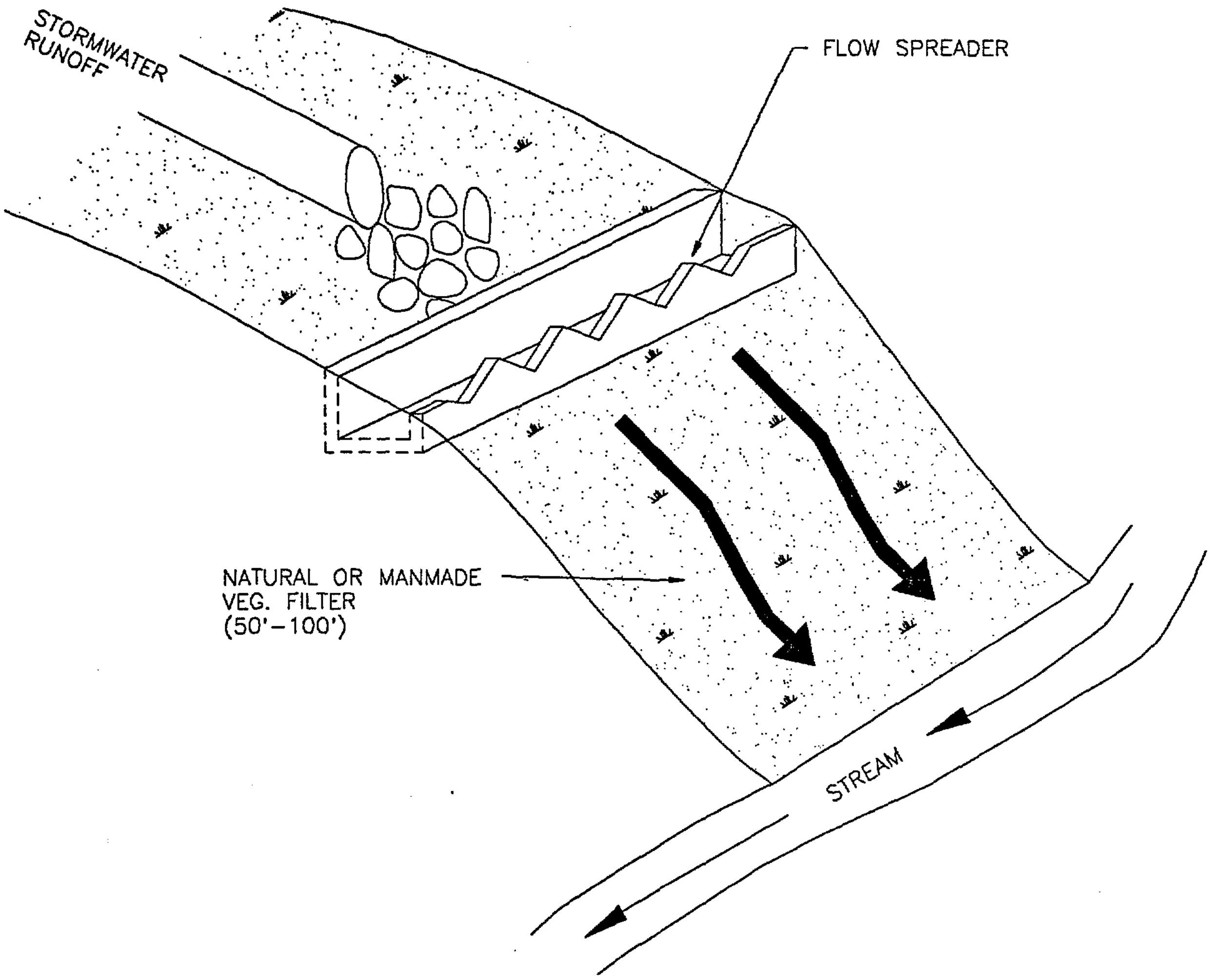


EXAMPLE 2

Figure 6 Effective Buffer Length Determination for Trap Efficiencies (T_R) of 75 to 99 percent (Buffer Length for coarse silt.)

Source: Wong and McCuen, 1982

FIGURE 7



VEGETATIVE FILTER DETAIL

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5.5 Maintenance Requirements

Maintenance is a vital factor in maintaining an adequate vegetative erosion control cover. See Table 10 to obtain the maintenance fertilization program for permanent seedings. When the filter strip is established and is functioning properly, fertilization should be minimized.

5.5.1 Irrigation: If soil moisture becomes deficient, irrigate to prevent loss of stand of protective vegetation.

5.5.2 Repairs: Inspect all seeded areas for failures and make necessary repairs, replacements, and reseedings within the current planting season, if possible.

(1) If a stand is inadequate for erosion control, overseed and fertilize using half of the rates originally applied.

(2) If stand is over 60 percent damaged, reestablish following original lime, fertilizer, seedbed preparation, and seeding recommendations.

Maintenance costs for filter strips depend on various factors such as length of the filter strip, type of vegetation, frequency of mowing, and can vary for each filter strip.

5.6 Life Expectancy

Filter strips are similar to grassed swales. As a BMP, filter strips can last for a long time, probably 10 to 20 years, if ideal conditions are maintained on site. Life expectancy of the filter strip may be only six (6) months if evenly distributed sheet flow and uniform, dense, and vigorous vegetation is not maintained.

5.7 Cost

Vegetative filter strip costs are similar to grassed swales costs. Costs are minimal when existing grass or meadow area is reserved at the site before development begins. If a filter strip is used as an on-site erosion control practice during the construction phase of development, it can be rehabilitated with a small expenditure after the development is complete.

5.8 Construction Specifications

5.8.1 Site Preparation

(1) Install needed erosion and sediment control practices such as silt fences, dikes, and contour ripping, erosion stops, channel lines, sediment traps, and sediment basins.

(2) If grading is required and topsoil is suitable for use, remove and stockpile the topsoil.

Note: Topsoil salvaged from the existing site may often be used but it should meet the same standards as set forth in these specifications. The depth of topsoil to be salvaged shall be six (6) inches unless the depth described as a representative profile for the particular soil type in the soil survey is less than six (6) inches, in which case the lesser depth shall be removed.

(3) Grade as needed and feasible to permit the use of conventional equipment for seedbed preparation, seeding, mulch application, anchoring, and maintenance.

(4) Liming: Where the subsoil is either highly acid or composed of heavy clays, ground dolomite limestone shall be spread at the rate of two (2) tons per acre (100 pounds per 1,000 square feet). Lime shall be distributed uniformly over designated areas and worked into the soil in conjunction with tillage operations as described in the following procedures.

(5) Tilling: After the area to be topsoiled has been brought to grade, and immediately prior to dumping and spreading the top-soil, the subgrade shall be loosened by discing and by scarifying to a depth of at least three (3) inches to permit bonding of the topsoil to the subsoil. The track of a bulldozer moving perpendicular to the contour will create small horizontal check dams to help prevent top soil from sliding down the slope.

5.8.2 Soil Preparation and Amendments

(1) Materials: Topsoil shall be loamy sand, sandy loam, loam, or silt loam, only, and in that respective order of preference. It shall not have a mixture of contrasting textured subsoil and contain no more than five (5) percent by volume of cinders, stones, slag, coarse fragments, gravel, sticks, roots, trash, or other extraneous materials larger than 1-1/2 inches in diameter. Topsoil must be free of plants or plant parts of bermudagrass, quackgrass, Johnsongrass, nutsedge, poison ivy, or Canada thistle. All topsoil shall be tested by a recognized laboratory for organic matter content, pH and soluble salts. A pH of 6.0 to 7.5 and an organic content of not less than 1.5 percent by weight is required. If the pH value is less than 6.0, lime shall be applied and incorporated with the topsoil to adjust the pH to 6.5 or higher. Topsoil containing soluble salts greater than 500 parts per million shall not be used.

No sod or seed shall be placed on soil which has been treated with soil sterilants or chemicals used for weed control until sufficient time according to manufacturer guidelines has elapsed to permit dissipation of toxic materials.

Note: Topsoil substitutes or amendments as approved by a qualified agronomist or soil scientist may be used in lieu of natural topsoil.

(2) Grading: The topsoil shall be uniformly distributed and tracked and shall be a minimum compacted depth of six (6) inches. Spreading shall be performed in such a manner that sodding or seeding can proceed with a minimum of additional soil preparation and tillage. Any irregularities in the surface resulting from topsoiling or other operations shall be corrected in order to prevent the formation of depressions or water pockets. Topsoil shall not be placed while in a

frozen or muddy condition, when the subgrade is excessively wet, or in a condition that may otherwise be detrimental to proper grading and seedbed preparation.

(3) Lime and fertilize according to soil tests: Lime and fertilizer needs can be determined by a qualified soil testing laboratory.

(4) In lieu of soil tests, apply 1,000 pounds of 10-10-10 (basic fertilizer) or equivalent per acre if ureaform fertilizer is not used, and 600 pounds of 10-10-10 or equivalent per acre if ureaform fertilizer is used. Apply the lime and basic fertilizer before seeding and harrow or disc uniformly into the soil to a minimum depth of three (3) inches on slopes flatter than three to one (3:1). On slopes steeper than three to one (3:1), the lime and fertilizer shall be worked in as well as possible. On sloping land, the final harrowing or discing operation should be on the contour wherever feasible. No attempt should be made to drag any disced area to make the soil surface very smooth after discing. When the 600 pounds per acre rate of 10-10-10 basic fertilizer application is used, then at the time of seeding, apply 30-0-0 ureaform fertilizer at a rate of 400 pounds per acre.

Note: The slow release ureaform fertilizer will supply nitrogen over a longer period of time and will result in a healthier stand of grass.

5.8.3 Seeding

(1) Select a mixture from Table 9.

(2) Apply seed uniformly with a cyclone seeder, drill, cultipacker seeder, or hydroseeder (slurry includes seed and fertilizer) on a firm, moist, seedbed. Maximum seeding depth should be 1/4-inch on clayey soils, when using other than hydroseeder method of application.

Note: If hydroseeding is used and the seed and fertilizer is mixed, they will be mixed on site and the seeding shall be immediate without interruption.

Locally accepted and approved seeding mixtures can also be used. CBLAD recommends the permanent seeding guidelines contained in the Virginia Erosion and Sediment Control Handbook.

Table 9 - Permanent Seeding and Seeding Dates						
Mix No.	Seeding Mixtures (Use Certified Seed if Available)	Lbs/ Acre	Lbs/ 1000 Sq. Ft.	Coastal Plain		
				2/1 - 4/30	5/1 - 8/14	8/15 - 10/31
1	'Kentucky 31' Tall Fescue*	60	1.38	X	-	X
2	'Kentucky 31' Tall Fescue* 'Boer' 'Lehmans' (a) Weeping Lovegrass	60 2	1.38 .05	-	X	-
3	'Kentucky 31' Tall Fescue* 'Korean' lespedeza (b) inoculated (h)	50 15	1.15 .34	X	-	-
4	'Kentucky 31' Tall Fescue* 'Interstate' Serices lespedeza (b)(h)	40 20	.92 .46	X	-	X
5	'Kentucky 31' Tall Fescue* Birdsfoot trefoil, inoculated(h)	40 10	.92 .23	-	-	-
6	'Kentucky 31' Tall Fescue (75%) Redtop (5%) Canada Bluegrass (10%) Kentucky Bluegrass (10%)(e)	90	2	X	-	X
7	Kentucky Bluegrass (50%) "Pennlawn' Creeping Red Fescue (40%) Redtop (10%)	90	2	X	-	X
8	<u>Droughty Areas</u> 'Kentucky 31' Tall Fescue* Redtop	30 5	.69 .11	X	-	X
9	Weeping lovegrass Serecia lespedeza(b) inoculated(h)	2 20	.05 .46	X	X	-
10	<u>Poorly Drained Areas</u> 'Kentucky 31' Tall Fescue*	30	.69	X	-	X
11	Reed canarygrass (c)	10	.23	-	-	X
12	<u>Shaded Areas</u> 'Kentucky 31' Tall Fescue*	60	1.38	X	-	X
13	Red Fescue 'Jamestown' or 'Pennlawn'	40	.92	X	-	X
14	<u>Lawns & High Maintenance Areas</u> 'Plush', 'Birka', 'Parade', 'Vantage' 'Columbia', 'Merion', 'Adelphi', 'South')** 'Dakota', 'Kenblue', Kentucky Bluegrass, Red Fescue, 'Pennlawn' or 'Jamestown'	90 10	2 .23	X	-	X
15	'Kentucky 31' Tall Fescue* (g)	220-260	5-6	X	X(f)	X

Source: Maryland Standards and Specifications for Stormwater Management Infiltration Practices.
* Certified Seed Only
** Three (3) varieties at 30 lb. each to make the 90 lb. mix.

Footnotes for Table 9.

(a) Use Weeping lovegrass to provide a stand of grass for erosion control during summer.

(b) Use hulless seed.

(c) Preferable to seed in fall with seed from current year's crop.

(d) All mixtures except 2 and 9 may be seeded during winter months in an emergency if two (2) tons per acre of a well-anchored mulch is used.

(e) Approved State Highway Administration Mixtures.

(f) Can be seeded during this period if irrigation water is used. Use two (2) tons per acre of well-anchored straw mulch.

(g) Can use ten (10) percent Kentucky bluegrass.

(h) Leguminous Seeds. All leguminous seeds shall be inoculated or treated with unexpired approved culture for the specific legume in the proper proportions as specified on the package label. The inoculant shall be stored at room temperatures, out of direct sunlight and away from heating units. When seeding dry with mechanical seeders, the following method of mixing the inoculant with the seed shall be followed. The culture in powder form is preferred and shall be thoroughly mixed with the seed by using a very small quantity of water; just enough to dampen the seeds before the culture is powdered on. The leguminous seed is then mixed with the other seeds of the formula. Seeds inoculated with the powder shall be sown within 48 hours after treatment. Seeds inoculated with the liquid culture shall be sown within 24 hours after treatment. Inoculated seed not used within these time periods shall be reinoculated. Inoculant and seed treated with inoculant shall not be exposed to sunlight for more than one (1) hour prior to seeding. When seed is applied by hydraulic seeders, ten (10) times the quantity of inoculant recommended for dry leguminous seed application shall be used. Inoculated seed shall not be held in a slurry with fertilizer for more than one (1) hour, otherwise reinoculation will be required before applying the seed.

5.8.4 Mulching

Mulch materials are listed in order of their effectiveness. Mulch mattings are normally only used on critical areas such as waterways or steep slopes.

5.8.5 Materials and Amounts

(1) Mulch mattings: Mattings such as jute or excelsior blanket shall be stapled to the surface in waterways and on steep slopes. Lighter materials of paper, plastic, and cotton mulch mattings may be used where erosion hazard is not severe. If the area is to be mowed, do not use metal staples.

(2) Straw: Straw shall be unrotted small grain applied at the rate of 1-1/2 to two (2) tons per acre, or 70 - 90 (two bales) pounds per 1,000 square feet. Mulch materials shall be relatively free of all kinds of weeds and shall be free of prohibited noxious weeds such as thistles, Johnsongrass, and quackgrass.

Spread uniformly by hand or mechanically. For uniform distribution of hand spread mulch, divide area into approximately 1,000 square foot sections and place 70-90 pounds of mulch in each section.

(3) Wood cellulose fiber: Mulch at the rate of 1,500 pounds per acre or 35 pounds per 1,000 square feet. Wood cellulose fiber may be applied by hydroseeding.

5.8.6 Mulch anchoring: Anchoring shall be accomplished immediately after mulch placement to minimize loss by wind or water. This may be accomplished by one of the following methods, (listed by preference) depending upon size of area, erosion hazard, and cost. On sloping land, practice No. 1 below, should be accomplished on the contour whenever possible. Contouring of all operations applies to all straw and to wood chip practices on more critical sites, except "tracking" should be done up and down the slope with 1-1/2 inch cleat marks running across the slope.

(1) Mulch anchoring tool and tracking: A mulch anchoring tool is a tractor drawn implement designed to punch and anchor mulch into the surface two (2) inches of the soil. This practice affords maximum erosion control but is limited to flatter slopes where equipment can operate safely. "Tracking" is primarily used on three to 1 (3:1) or steeper cut and fill slopes to cut and mulch into the soil by 1-1/2 inch track cleats of a bulldozer making groves across the slope.

(2) Mulch netting: Staple lightweight biodegradable paper, plastic, or cotton nettings over the mulch according to the manufacturer's recommendations. Netting is usually available in rolls four (4) feet wide and up to 300 feet long.

(3) Liquid Mulch Binders: Applications of liquid binders should be heavier at edges where wind catches mulch, in valleys, and at crests of banks. The remainder of the area should be uniform in appearance. Caution should be used with asphalt in residential and similar areas.

(a) Cutback asphalt - rapid curing (RC-70, RC-250, and RC-800) or medium curing (MC-250 or MC-800). Apply five (5) gallons per 1,000 square feet or 200 gallons per acre on flat areas and on slopes less than eight (8) feet high. On slopes eight (8) feet or more high, use eight (8) gallons per 1,000 square feet or 348 gallons per acre.

(b) Emulsified asphalt - (SS-1, CSS-1, CMS-2, MS-2, RS-1, RS-2, CRS-1, and CRS-2). Apply five (5) gallons per 1,000 square feet or 200 gallons per acre on flat areas and on slopes less than eight (8) feet high. On slopes eight (8) feet or more high, use eight (8) gallons per 1,000 square feet or 348 gallons per acre.

All asphalt designations are from the Asphalt Institute Specifications.

(c) Synthetic binders: Synthetic binders such as Acrylic DI (Agri-Tax, DCA-70, Petroset or Terra Tac may be used at rates recommended by the manufacturer to anchor mulch material.

(4) Wood Cellulose Fiber: Wood cellulose fiber may be used for anchoring straw. The fiber binder shall be applied at a net dry weight of 750 pounds/acre. The wood cellulose fiber shall be mixed with water and the mixture shall contain a maximum of 50 pounds of wood cellulose fiber per 100 gallons.

(5) Peg and Twine: Drive eight- to ten-inch (8" to 10") wooden pegs to within two (2) to three (3) inches of the soil surface every four (4) feet in all directions. Stakes may be driven before or after applying mulch. Secure mulch to soil surface by stretching twine between pegs in a criss-cross within a square pattern. Secure twine around each peg with two (2) or more complete turns.

Note: All names given above are registered trade names. This does not constitute a recommendation of these products to the exclusion of other products.

5.8.7 Irrigation

If soil moisture is deficient, supply new seedings with adequate water for plant growth until they are firmly established. This is especially true when seedings are made late in the planting season, in abnormally dry or hot seasons, or on adverse sites.

Table 10 - Maintenance Fertilization for Permanent Seedings
Use Soil Test Recommendations or Rates Shown Below

Mixture No.	Seeding Mixture	Formulation	Lbs. Per Acre	Lbs. Per 1,000 Sq.Ft.	Time	Mowing
1,2,3,7,8, 10	Tall fescue makes up 70% or more of cover	10-10-10 or 30-0-0 10-10-10	500 400 600	11.5 9.2 13.8	Yearly or as needed Fall Yearly or as needed	* Not closer than 3" if occasional mowing is desired.
4,5	Fairly uniform stand of tall fescue and sericea lespedeza, or birdsfood trefoil	5-10-10	500	11.5	Fall the year following establishment and every 4-5 years thereafter.	Not required. Not closer than 4" if occasional mowing is desired,and then in fall after seed has matured.
11	Weeping lovegrass and sericea lespedeza. Fairly uniform plant distribution	5-10-10	500	11.5	Spring the year following establishment and every 3-4 years thereafter	Not required. Not closer than 4" if occasional mowing is desired and then in fall after sericea has matured.
9,12,13,14	Red fescue	20-10-10	250	5.8	September, 30 days later	Mow no closer than 2" for red fescue and Kentucky bluegrass; and no closer than 3" for fescue.
15,17	Kentucky bluegrass-red fescue mixture; 'Ky-31' tall fescue	20-10-10 20-10-10 20-10-10	250 250 100	5.8 5.8 2.3	December, May 20-June 30 if needed	

Source: Maryland Standards and Specifications for Soil Erosion and Sediment Control.
Note: Under Mixture No., refer to Table 9.

6. DRY WELL

6.1 Description

A dry well is an excavated pit lined with engineering filter fabric and backfilled with stone aggregate. The dry well is generally a much smaller structure than an infiltration trench. Inflow to the dry well is mostly through an inflow pipe. A typical dry well schematic is shown on Figure 8 and a dry well detail is shown on Figure 9.

6.2 Applicability

A dry well is generally used to capture the runoff from roof top areas of less than one (1) acre in surface area. This BMP is used to store runoff from residential, commercial, and industrial buildings.

6.3 Design Criteria

6.3.1 Soil Permeability

Soil textural classes with infiltration rates greater than, or equal to, 0.27 inches per hour should be used for the design of dry wells. This infiltration rate is associated with soil textural groups of sand, loamy sand, sandy loam, loam, and silt loam. The infiltration rate of the underlying soil where the well is located is the major limiting factor in the selection and feasibility of the dry well as a BMP.

6.3.2 Depth of Well

The final infiltration rate of the soil below the dry well determines the maximum allowable well depth. A well with a grass covered surface should have at least one (1) foot of overlying soil above the stone aggregate reservoir. In case a dry well is installed under a driveway or a patio deck, the depth of overlying soil should be considered zero. The surface area of the well can be minimized by making the dry well as deep as feasible. On the other hand, the dry well can be made shallow and broad. The increased surface area of the bottom of the dry well increases exfiltration and provides more area for soil filtering of pollutants. The larger well bottoms also help in reducing clogging at the soil/filter cloth interface by providing exfiltration over a wide area.

6.3.3 Groundwater Table

The seasonal high groundwater table should be located at least two (2) to four (4) feet below the bottom of the well. The soil permeability (infiltration rate) and the groundwater table are the two parameters which determine the maximum allowable depth of the well.

6.3.4 Proximity to Wells and Foundations

Dry wells should be located at least 100 feet upgradient from any drinking water supply well to minimize the possibility of groundwater contamination. Also the dry well should be located at least ten (10) feet downgradient and 100 feet upgradient from building foundations.

6.3.5 Design Storm

Dry wells can be designed for a storm of a specific recurrence interval or for the first flush runoff volume. If for first flush, storage volume can be sized based on 0.5 inches of runoff per acre of rooftop area.

Dry wells are normally designed as water quality facilities for runoff generated from rooftops. As such, a significant portion of the runoff volume from the site will bypass the dry well and is not infiltrated. Additional BMPs should be installed to infiltrate storm runoff from other impervious areas on the site.

6.3.6 Storage Time/Maximum Draining Time

All dry wells should be designed to drain within a maximum time of three (3) days (72 hours), or a minimum time of two (2) days (48 hours). These values are derived from existing literature. The Virginia Stormwater Management Regulations recommend two (2) days (48 hours).

6.3.7 Stone Aggregate

The stone aggregate which fills the dry well forms the reservoir through which the storm runoff passes and is filtered. The aggregate material should be clean, washed stone. Wash run gravel is preferred. The City of Virginia Beach recommends using James River stone as aggregate. The clean washed stone aggregate should have a maximum diameter of three (3) inches and a minimum diameter of one (1) inch. Void spaces for the stone aggregate should fall within the range of 30 to 40 percent. A table showing open graded coarse aggregates is included in the Appendix.

6.3.8 Observation Well

An observation well should be installed in the dry well. The observation well can be a perforated PVC pipe, four (4) to six (6) inches in diameter. The pipe should be located in the center of the well with the bottom resting on a plate. The top of the observation well should be capped to prevent vandalism.

The observation well helps in monitoring the function of this BMP. The water level in the observation well should be measured after a storm event. If the dry well does not drain completely after three (3) days, the well is not functioning properly and remedial steps may need to be taken to improve its performance.

6.3.9 Runoff Filtering

Since the dry well is designed to capture the runoff from rooftops, screens should be placed at the top of the roof downdrains to prevent leaves and other debris from entering the dry well.

6.3.10 Overflow Requirements

The overflow path of the surface runoff exceeding the capacity of the dry well should be evaluated. A surcharge pipe above the dry well should be installed to allow drainage in extreme events.

6.4 Design Examples

A one-acre (1 ac.) lot is to be developed for constructing a house with a roof-top area of 2,000 square feet. The soil borings on the site indicate that the soil is silt loam with an infiltration rate of 0.27 inches per hour. The depth of the seasonal high water table is determined to be five (5) feet deep.

Design a dry well to capture the runoff from a one-inch (1") rainfall and for first flush. The runoff from a one-inch (1") rainfall is computed to be 0.30 inch. The depth of soil over the dry well is one (1) foot and runoff depth from area over dry well is computed to be 0.03 inch.

DESIGN OF DRYWELL FOR 1" RAINFALL

Project: 6.4 DESIGN EXAMPLE FOR 2000 SQ.FT. HOUSE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.27

Maximum Allowable Storage Time of Well(Hrs): 72

Void Ratio Well Medium: 0.4

Depth to Seasonal High Groundwater Table(Ft): 5.0

Min. Dist. from Well bottom to Groundwater Table(Ft): 2.0

***** Maximum Well Depth(Ft): 3.0 *****

----->>> Design Input Parameters <<<-----

Rooftop Area of Runoff to Dry Well(Sq Ft): 2000

Runoff Depth from Rooftop Area (In): 0.30

Runoff Depth from Area over Dry Well(In): 0.03

Depth of Well(Ft): 3.0

Depth of Soil Overlying Dry Well(Ft): 1.0

Rainfall(In): 1.00

Dry Well Filling Time(Hrs): 1

Water Capacity of Overlying Soil(In/In): 0.17

***** Well Area(Sq Ft): 41 *****

DESIGN OF DRYWELL FOR FIRST FLUSH

Project: 6.4 DESIGN EXAMPLE FOR 2000 SQ.FT. HOUSE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.27

Maximum Allowable Storage Time of Well(Hrs): 72

Void Ratio Well Medium: 0.4

Depth to Seasonal High Groundwater Table(Ft): 5.0

Min. Dist. from Well bottom to Groundwater Table(Ft): 2.0

***** Maximum Well Depth(Ft): 3.0 *****

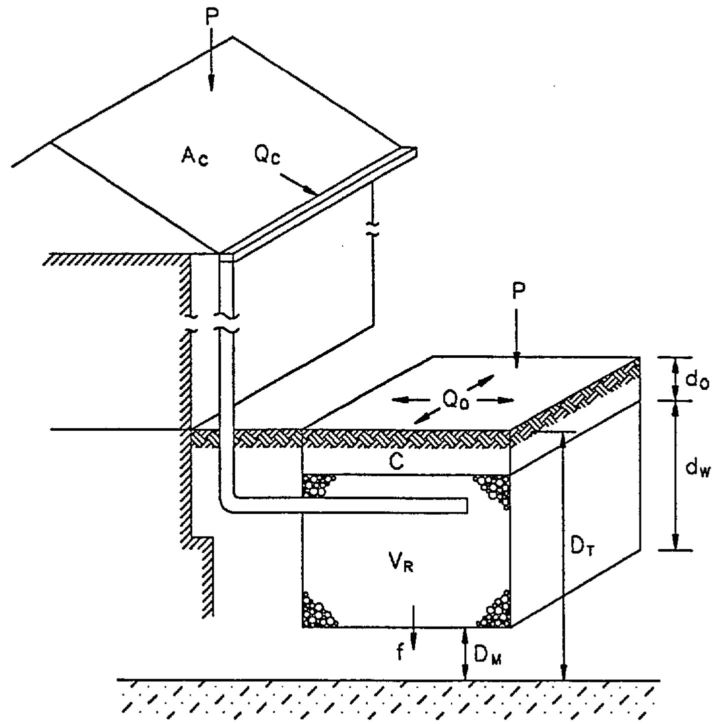
----->>> Design Input Parameters <<<-----

Rooftop Area of Runoff to Dry Well(Sq Ft): 2000

Runoff Depth from Rooftop Area (In): 0.50

Depth of Well(Ft): 3.0

***** Well Area(Sq Ft): 69 *****



f = Infiltration Rate(In/Hr)

T_S = Maximum Allowable Storage Time of Well(Hrs)

V_R = Void Ratio Well Medium

D_T = Depth to Seasonal High Groundwater Table(Ft)

D_M = Min Dist from Well bottom to Groundwater Table(Ft)

A_C = Rooftop Area of Runoff to Dry Well(Sq Ft)

Q_C = Runoff Depth from Rooftop Area (In)

Q_O = Runoff Depth from Area over Dry Well(In)

d_W = Depth of Well(Ft)

d_O = Depth of Soil Overlying Dry Well(Ft)

P = Rainfall(In)

T = Dry Well Filling Time(Hrs)

C = Water Capacity of Overlying Soil(In/In)

For First Flush Design:

$$\text{Area of Well} = \frac{A_C \frac{Q_C}{12}}{V_R\, d_W}$$

$$\text{Area of Well} = \frac{A_C \frac{Q_C}{12}}{V_R\, d_W - \frac{P}{12} + \frac{Q_O}{12} + d_O C + \frac{f}{12} T}$$

NOTE: IF $d_O C > \frac{P}{12} - \frac{Q_O}{12}$ SET $- \frac{P}{12} + \frac{Q_O}{12} + d_O C = 0$

DRY WELL SCHEMATIC

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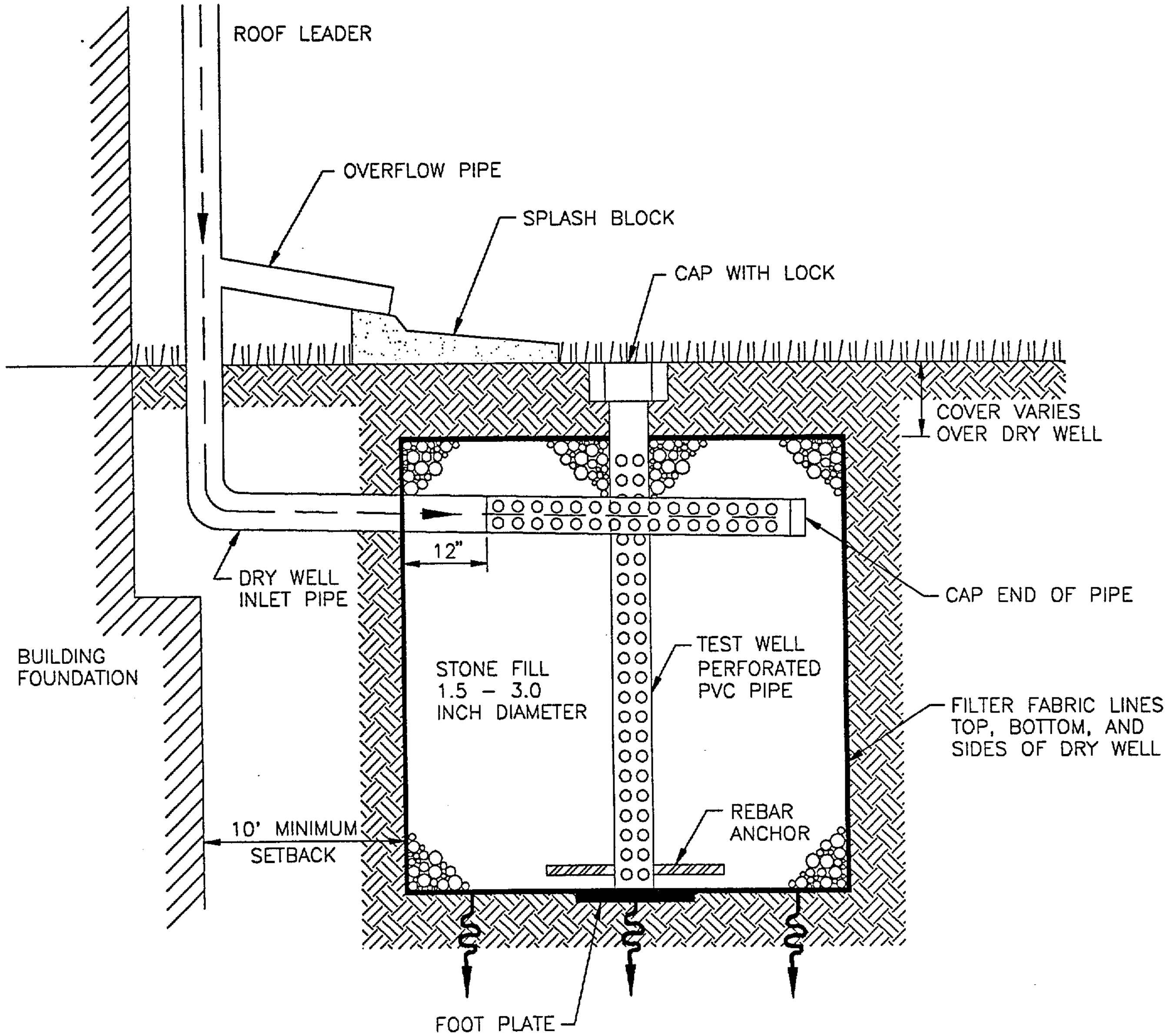
FIGURE 9



DRY WELL DETAIL

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6.5 Maintenance Requirements

All dry wells are prone to clogging even if they are properly designed and constructed. However, routine maintenance requirements for dry wells are minimal. The major maintenance item is to clean the roof leader of tree leaves, pine needles, any separated shingle particles, and other debris from the roof. The cleaning of gutters should be performed on an as needed basis.

6.5.1 Inspection

The dry well should be inspected several times in the early months of operation. It should be inspected once a month initially for a period of six (6) months. The inspection should be conducted after large or frequent storms to determine the water level in the observation well. A log book should be maintained to indicate the inspection visits and the rate at which the dry well dewaters or exfiltrates. Once the performance characteristics of the dry well have been determined, monitoring can be performed on a semi-annual or annual basis.

6.5.2 Non-Routine Maintenance

Despite careful design, construction, and maintenance, some dry wells will get clogged and need rehabilitation. Clogging in dry wells is most likely to occur near the top of the dry well, between the interface of stone and filter fabric. Surface clogging can be fixed by carefully removing the top layer of vegetation and stone, removing the clogged filter fabric, installing new filter fabric, and cleaning or replacing the top layer of stone. Clogging can also occur at the bottom of the dry well at the filter fabric/soil interface. Rehabilitation of the dry well then requires the removal of the top layer of vegetation and stone, the filter fabric, the entire stone aggregate reservoir, and the bottom filter fabric layer. Before the dry well is reconstructed, the subsoil layer should be scarified to promote better infiltration.

6.5.3 Total Maintenance Costs

Rehabilitation of dry wells with complete reconstruction will cost the same as the initial construction cost. Partial dry well rehabilitation may cost approximately 20 percent of the initial construction cost. An annual set-aside of five (5) to ten (10) percent of the initial construction cost should be accumulated to cover routine/non-routine maintenance expenditures. These estimates are based on existing information and may vary from site to site and differ for each jurisdiction. Reliable maintenance costs and life expectancies of dry wells will become more accurate with experience and time.

6.6 Life Expectancy

Dry well as a BMP has not been in use for a long enough time to determine its life expectancy. More monitoring of existing facilities needs to be done before reliable information for life expectancy can be determined. Based on experience in the State of Maryland, a dry well may function properly anywhere from six (6) months to two (2) years. Proper construction, inspection, and regular maintenance in terms of removing debris, leaves, and other materials from rooftop gutters may enhance the useful life of drywell as a BMP.

6.7 Cost

A general planning estimate for infiltration trench costs can be obtained by using the following relationship:

$$C = 32.7\, V_s^{0.63}$$

where C = construction cost in 1990 dollars

V_s = storage volume in cubic feet.

The above planning equation to estimate dry wells costs should not be used for storage volumes greater than 10,000 cubic feet. Costs associated with other appurtenances are not included in the above relationship. An additional 25% should be added to the above derived planning cost to cover contingency costs.

6.8 Construction Specifications

6.8.1 Trench Preparation

Excavate the dry well to the design dimensions. Excavated materials should be placed away from the excvated sides for wall stability. Large tree roots must be trimmed flush with the sides in order to prevent fabric puncturing or tearing during subsequent installation procedures. The side walls of the dry well should be roughened where sheared and scaled by heavy equipment.

6.8.2 Fabric Laydown

The filter fabric roll must be cut to the proper width prior to installation. This width must include sufficient material to conform to dry well perimeter irregularities and for a six-inch minimum top overlap. Place the fabric roll over the dry well and unroll a sufficient length to allow placement of the fabric down into the dry well. Stones or other anchoring objects should be placed on the fabric at the edge of the dry well to keep the lined dry well open during windy periods. When overlaps are required between rolls, the upstream roll should lap a minimum of two (2) feet over the downstream roll in order to provide a shingled effect. The overlap ensures fabric continuity and ensures that the fabric conforms to the excavation surface during aggregate placement and compaction.

A partial list of suggested filter fabric brands is listed in Table 11.

Table 11 - Approved Geo-Textiles For Use in Dry Wells	
Mirafi 140-N	Note: This is a partial list of acceptable filter fabrics available from suppliers for use in infiltration trenches. The use of a brand name does not constitute an endorsement by HRPDC of any particular product or company.
Supac 4NP, 4.5NP, 5NP, and 8NP	
Typar 3401	
AMOCO 4545	
EXXON Geo-textiles No. 125D, 130D, and 150D	
TerraTex SD	
Source: "Controlling Urban Runoff" - Metropolitan Washington Council of Governments.	

6.8.3 Stone Aggregate Placement and Compaction

The stone aggregate should be placed in lifts and compacted using plate compactors. As a rule of thumb, a maximum loose lift thickness of 12 inches is recommended. The compaction process ensures fabric conformity to the excavation sides, thereby reducing the potential for soil piping, fabric clogging, and settlement problems.

6.8.4 Overlapping and Covering

Following the stone aggregate placement, the filter fabric should be folded over the stone aggregate to form a six-inch (6") minimum longitudinal lap. The desired fill soil or stone aggregate should be placed over the lap at sufficient intervals to maintain the lap during subsequent backfilling.

6.8.5 Contamination

Care shall be exercised to prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate must be removed and replaced with uncontaminated stone aggregate.

6.8.6 Voids Behind Fabric

Voids should be avoided between the fabric and excavation sides. Removing boulders or other obstacles from the walls is one source of such voids. Natural soils should be placed in these voids at the most convenient time during construction but prior to installing fabric to ensure fabric conformity to the excavation sides. Soil piping, fabric clogging, and possible surface subsidence will be avoided by this remedial process.

6.8.7 Unstable Excavation Sides

Vertically excavated walls may be difficult to maintain in areas where the soil moisture is high or where soft cohesive or cohesionless soils predominate. These conditions may require laying back of the side slopes to maintain stability; trapezoidal rather than rectangular cross sections may result.

6.8.8 Vegetative Buffer

A vegetative buffer of at least 20 feet wide (wider if possible) should be used to intercept surface runoff from all impervious areas.

6.8.9 Traffic Control

Heavy equipment and traffic shall be restricted from travelling over the infiltration areas to minimize compaction of the soil.

6.8.10 Observation Well

An observation well, as described in subsection 6.3.8 and Figure 9 should be provided. The depth of the well at the time of installation should be clearly marked on the well cap.

7. INFILTRATION TRENCH

7.1 Description

An infiltration trench is a shallow excavated pit, generally two (2) to ten (10) feet in depth, backfilled with coarse stone aggregate. Stormwater runoff is temporarily stored in the voids in the aggregate material and gradually infiltrates into the surrounding and underlying soil. Infiltration trenches are a viable BMP for permeable soils when the water table is two (2) to four (4) feet below the bottom of the trench.

Infiltration trenches can remove both soluble and particulate pollutants. Stormwater runoff is generally laden with sediments and coarse material which should be prevented from entering the trench. The runoff should enter the trench through a minimum 20-foot wide vegetative buffer strip for surface trenches or through structures such as water quality inlets or grit-oil separators for underground trenches. By capturing the sediments before the runoff enters the trench, the life of the trench can be increased.

An infiltration trench schematic is shown on Figure 10. Infiltration trench details are shown on Figures 11 and 12.

7.2 Applicability

Infiltration trenches are primarily on-site control BMPs and are generally applicable to small drainage areas (1 to 10 acres). This BMP can be installed in residential developments and open space areas as a surface trench, and in commercial areas as an underground trench with special inlets. An infiltration trench can also be installed under a grass swale.

7.3 Design Criteria

7.3.1 <u>Soil Permeability</u>

Soil textural classes with infiltration rates greater than or equal to 0.27 inches per hour should be used for the installation of infiltration trenches. This infiltration rate is associated with soil textural groups of sand, loamy sand, sandy loam, loam, and silt loam.

The infiltration rate of the underlying soil and the depth of the groundwater table are the major limiting factors in the selection and feasibility of the infiltration trench as a BMP.

7.3.2 Depth of Trench

The final infiltration rate of the soil below the infiltration trench determines the maximum allowable trench depth. A trench with a grass covered surface should have at least one (1) foot of overlying soil above the stone aggregate reservoir. The surface area of the trench can be minimized by making the trench as deep as feasible. The trench can also be made shallow and broad. The increased surface area of the bottom of the trench increases exfiltration rates and provides more area for soil filtering of pollutants. A larger trench bottom also helps in reducing clogging at the soil/filter cloth interface by providing exfiltration over a wide area.

7.3.3 Groundwater Table

The seasonal high groundwater table should be located at least two (2) to four (4) feet below the bottom of the trench. The soil permeability (infiltration rate) and the groundwater table are the two parameters which determine the maximum allowable depth of the trench.

7.3.4 Proximity To Wells and Foundations

Infiltration trenches should be located at least 100 feet upgradient from any drinking water supply well to minimize the possibility of groundwater contamination. Also the trenches should be located at least ten (10) feet downgradient and 100 feet upgradient from building foundations.

7.3.5 Design Storm

Infiltration trenches can be designed for a specific storm or for the first flush runoff volume. If for first flush, the trench storage volume can be sized based on 0.5 inches of runoff per impervious acre in the contributing site area.

Infiltration trenches are normally designed for water quality. As such, a significant portion of the runoff volume (storms producing more than 0.5 inches of runoff) will bypass the trench and is not infiltrated.

7.3.6 Storage Time/Maximum Draining Time

All infiltration trenches should be designed to drain within a maximum time of three (3) days (72 hours), or a minimum time of two (2) days (48 hours). These values are derived from literature. The Virginia Stormwater Management Regulations recommend two (2) days (48 hours).

7.3.7 Stone Aggregate

The stone aggregate which fills the infiltration trench forms the reservoir through which the storm runoff passes and is filtered. The aggregate material should be clean, washed stone. Wash run gravel is preferred. The City of Virginia Beach recommends using James River stone as aggregate. The clean washed stone aggregate should have a maximum diameter of three (3) inches and a minimum diameter of one (1) inch. Void spaces for the stone aggregate are normally within the range of 30 to 40 percent. A table showing open graded coarse aggregates is included in the Appendix.

7.3.8 Observation Well

An observation well shall be installed in the infiltration trench. The observation well should be a perforated PVC pipe, four (4) to six (6) inches in diameter. The pipe should be located in the center of the trench and the bottom should rest on a plate. The top of the well should be capped to prevent vandalism and tampering.

The observation well helps in monitoring the function of the trench. The water level in the trench should be measured after a storm event. If the trench does not completely drain after three (3) days, it indicates that the trench is not functioning properly and remedial steps may need to be taken to improve the performance.

7.3.9 Runoff Filtering

It is important to prevent any floatable material, settleable solids, grease, and oil from entering the infiltration trench. Runoff filtering devices such as vegetative filter strips (minimum of 20 feet) and water quality inlets can be used in front of the trench to prevent objectionable materials from entering the trench. All trenches with surface inlets shall be designed to capture the sediments and other material before the storm runoff discharges into the stone aggregate reservoir.

Infiltration trenches in combination with grass swales with check dams are feasible combinations to increase the volume of infiltration into the trench. In this alternative, the trench can be constructed under the swale with check dam to create a pool of water.

The sides of the trench should be lined with filter fabric to prevent the entry of sediments into the trench. The bottom of the trench, if constructed in good permeable soil, can be lined with a six-inch layer of sand or filter fabric.

In addition to the vegetative filter strip (minimum 20 feet), filter fabric placed one (1) or two (2) feet below the top of the trench can be used to prevent sediments from entering the trench.

7.3.10 Overflow Requirements

In all cases, the overflow path of storm runoff exceeding the capacity of the trench should be evaluated and accommodated. The trenches are designed to treat the first flush volume of runoff and control small drainage areas.

7.4 Design Examples

A one-acre (1 ac.) lot is to be developed for constructing a house with a roof-top area of 2,000 square feet. The soil borings on the site indicate that the soil is silt loam with an infiltration rate of 0.27 inches per hour. The depth of the seasonal high water table is determined to be five (5) feet deep.

Design an infiltration trench to capture the runoff from a one-inch (1") rainfall and for first flush. The runoff from a one-inch (1") storm is computed to be 0.30 inch.

DESIGN OF INFILTRATION TRENCH FOR 1" RAINFALL

Project: 7.4 DESIGN EXAMPLE FOR 2000 SQ.FT. HOUSE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.27

Maximum Allowable Storage Time of Trench(Hrs): 72

Void Ratio Trench Medium: 0.4

Depth to Seasonal High Groundwater Table(Ft): 5.0

Min. Dist. from Trench bottom to Groundwater Table(Ft): 2.0

******* Maximum Trench Depth(Ft): 3.0 *******

----->>> Design Input Parameters <<<-----

Increase in Runoff Depth(In): 0.30

Contributing Drainage Area(Sq Ft): 2000

Depth of Trench(Ft): 3.0

Amount of Rainfall(In): 1.00

Trench Filling Time(Hrs): 2

******* Trench Area(Sq Ft): 43 *******

DESIGN OF INFILTRATION TRENCH FOR FIRST FLUSH

Project: 7.4 DESIGN EXAMPLE FOR 2000 SQ.FT.HOUSE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.27

Maximum Allowable Storage Time of Trench(Hrs): 72

Void Ratio Trench Medium: 0.4

Depth to Seasonal High Groundwater Table(Ft): 5.0

Min. Dist. from Trench bottom to Groundwater Table(Ft): 2.0

***** Maximum Trench Depth(Ft): 3.0 *****

----->>> Design Input Parameters <<<-----

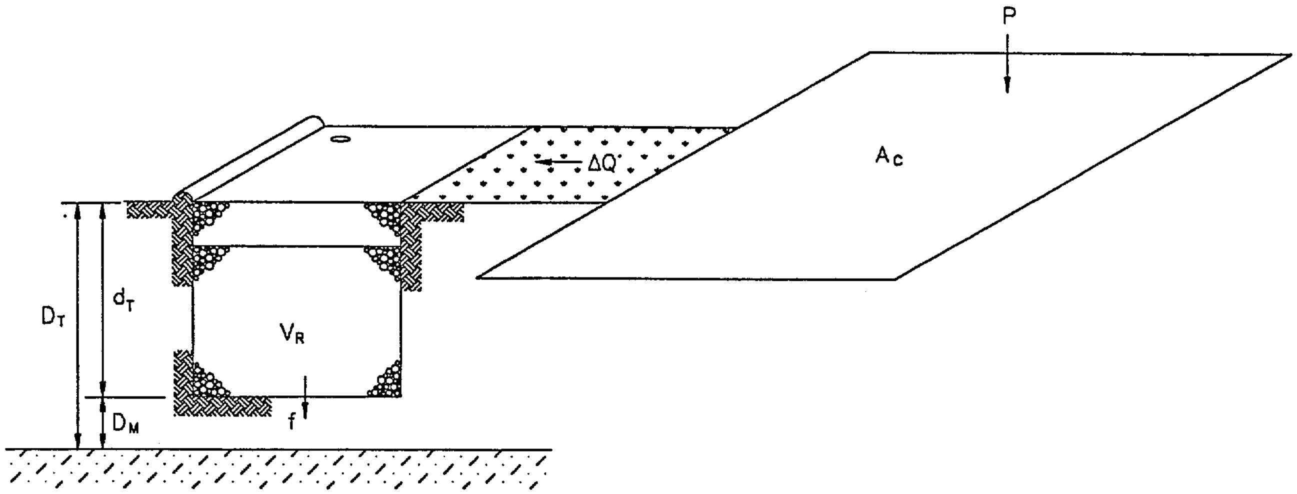
Increase in Runoff Depth(In): 0.50

Contributing Drainage Area(Sq Ft): 2000

Depth of Trench(Ft): 3.0

***** Trench Area(Sq Ft): 69 *****

FIGURE 10



f = Infiltration Rate(In/Hr)

T_S = Maximum Allowable Storage Time of Trench(Hrs)

V_R = Void Ratio Trench Medium

D_T = Depth to Seasonal High Groundwater Table(Ft)

D_M = Min Dist from Trench bottom to Groundwater Table(Ft)

A_C = Contributing Drainage Area(Sq Ft)

ΔQ = Increase in Runoff Depth(In)

d_T = Depth of Trench(Ft)

P = Amount of Rainfall(In)

T = Trench Filling Time(Hrs)

For First Flush Design:

$$\text{Area of Trench} = \frac{\frac{\Delta Q}{12} A_C}{V_R\, d_T}$$

$$\text{Area of Trench} = \frac{\frac{\Delta Q}{12} A_C}{V_R\, d_T - \frac{P}{12} + \frac{f}{12} T}$$

INFILTRATION TRENCH SCHEMATIC

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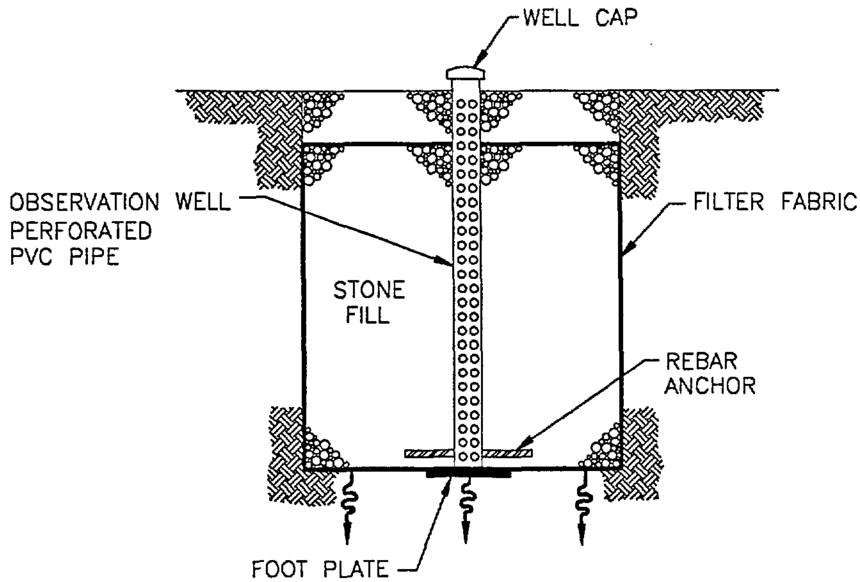
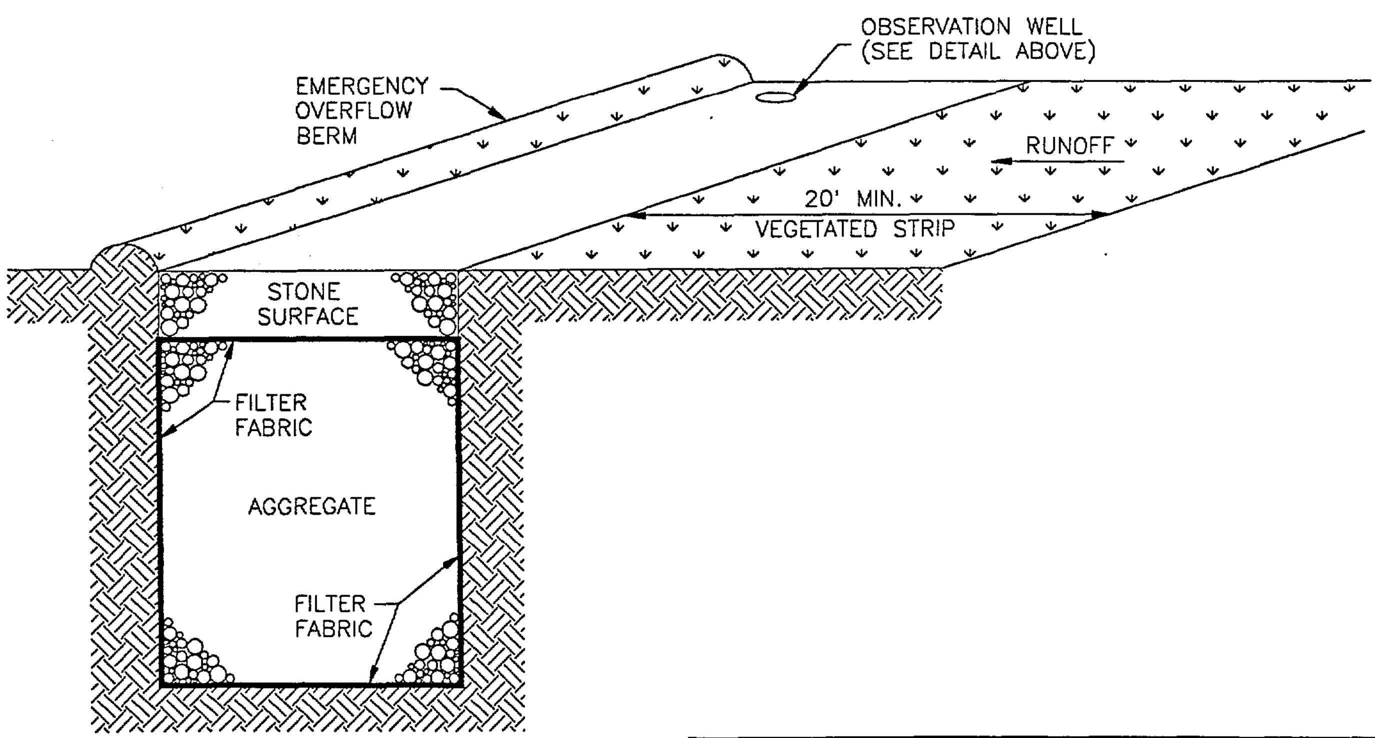
FIGURE 11



OBSERVATION WELL DETAIL



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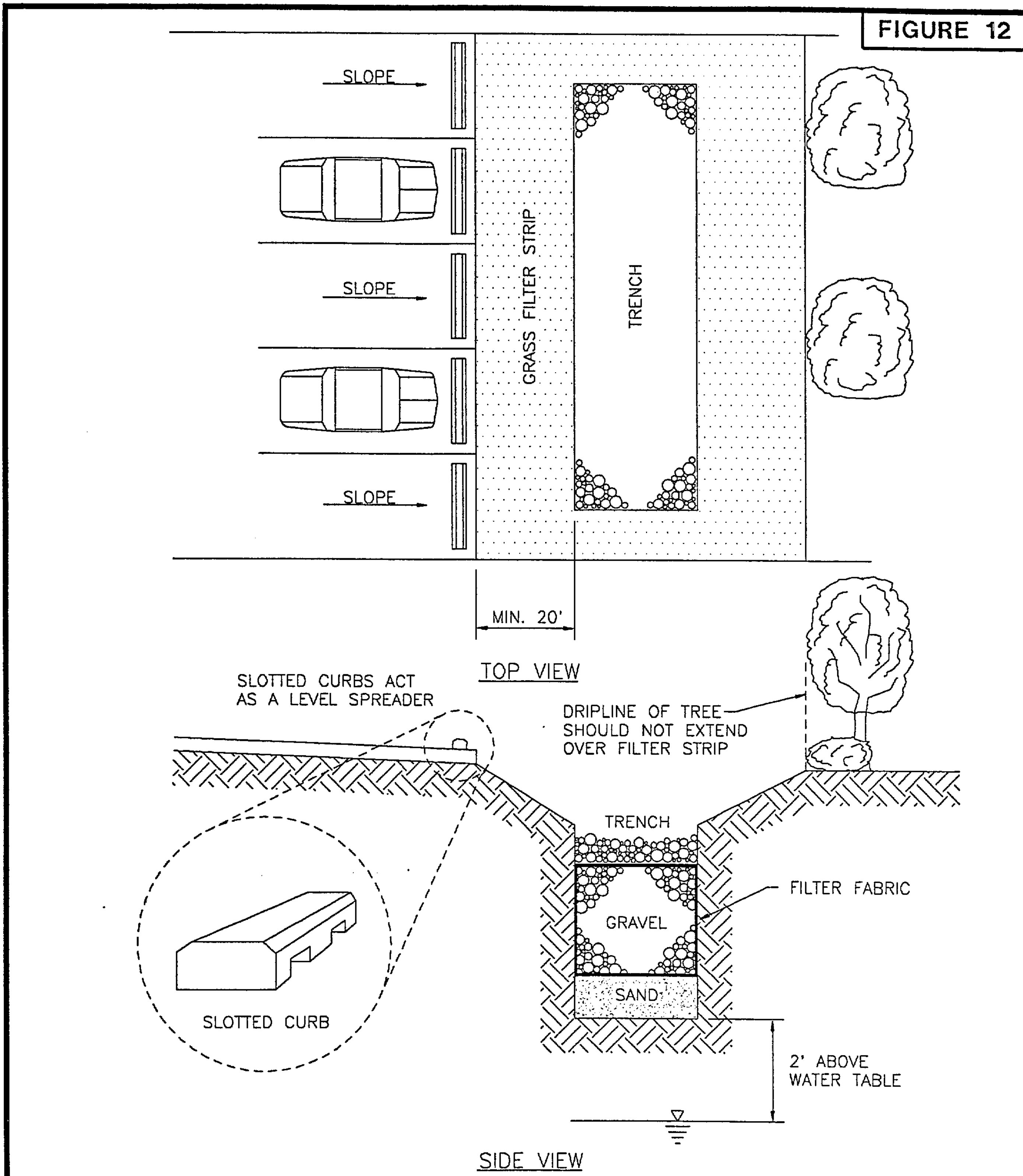

PARKING LOT PERIMETER TRENCH

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7.5 Maintenance Requirements

All Infiltration Trenches are prone to clogging by sediments even if they are properly designed and constructed. However, routine maintenance requirements for trenches are minimal.

7.5.1 Inspection

The trench should be inspected several times in the early months of operation. The inspection should be conducted after large or frequent storms to determine the water level in the observation well. A log book should be maintained to indicate the inspection visits and the rate at which the trench dewaters or exfiltrates. Once the performance characteristics of the trench have been determined, monitoring can be performed on a semi-annual or annual basis.

7.5.2 Sediment Removal

Infiltration trenches installed with water quality inlets for pre-treatment of runoff should be cleaned of sediments periodically, with a minimum interval of six (6) months. Built-up sediment, if not removed, will reduce the storage capacity of the water quality inlets. The removal of sediments and other objectionable material can be performed manually or by using a vacuum pump.

7.5.3 Buffer Maintenance

Vegetative filter strips should be mowed at least twice a year. Grass in the filter strip should not be mowed less than three (3) inches. Grass clippings should be either bagged or disposed of away from the trench. The condition of the vegetative filter strip should be inspected annually. Bare spots or eroded areas should be reseeded or re-sodded.

Trees should not be allowed to grow in the vicinity of the trench to prevent the roots from puncturing the filter fabric. Branches extending over the trench should be trimmed so that the tree leaves do not clog the trench.

7.5.4 Non-Routine Maintenance

Despite careful design, construction, and maintenance, some trenches will get clogged and need rehabilitation. In surface trenches, clogging is most likely to occur near the top of the trench, between the interface of stone and filter fabric. Surface clogging can be fixed by carefully removing the top layer of vegetation and stone, removing the clogged filter fabric, installing new filter fabric, and cleaning or replacing the top layer of stone. Clogging can also occur at the bottom of the trench at the filter fabric/soil interface. Rehabilitation of the trench then requires the removal of the top layer of vegetation and stone, the filter fabric, the entire stone aggregate reservoir, and the bottom filter fabric layer. Before the trench is reconstructed, the subsoil layer should be scarified to promote better infiltration.

Clogging of underground trenches should be alleviated in the same manner as surface trenches. If pavement or concrete are used as a surface layer for the trench instead of grass or vegetation, reconstruction of the trench can be costly and difficult.

7.5.5 Total Maintenance Costs

Rehabilitation of underground trenches including complete reconstruction of surface trenches may cost the same as the initial construction cost. Surface trench rehabilitation may cost approximately 20% of the initial construction cost. An annual set-aside of 5-10% of the initial construction cost for surface trenches and 10-15% for underground trenches may be required to cover routine/non-routine maintenance expenditures. These estimates are based on existing information and may vary from site to site and differ for each jurisdiction. Reliable maintenance costs and life expectancy of trenches will become more accurate with experience and time.

7.6 Life Expectancy

Infiltration trenches as BMPs have not been in use for a long enough time to determine life expectancy. More monitoring of existing facilities needs to be done before reliable information for life expectancy can be determined. An infiltration trench should be constructed after the contributing area has been stabilized to prevent the sediments in the runoff from clogging the trench. Based on experience in the State of Maryland, an infiltration trench may function properly anywhere from six (6) months to two (2) years. Proper construction, inspection, and regular maintenance may enhance the useful life of infiltration trench as a BMP. Life expectancy of infiltration trenches may vary from site to site depending upon the land use in the contributing area and the contents of the storm runoff.

7.7 Cost

A general planning estimate of infiltration trench costs can be made by using the following relationship:

$$C = 32.7\ V_s^{0.63}$$

where C = construction cost in 1990 dollars

V_s = storage volume in cubic feet.

This planning equation for estimating trench costs should not be used for storage volumes greater than 10,000 cubic feet. Costs associated with pretreatment of runoff, like vegetative filter strips or water quality inlets, and other appurtenances are not included in the above relationship. An additional 25% should be added to the above derived cost to cover contingency costs.

7.8 Construction Specifications

7.8.1 Timing

An Infiltration trench should not be constructed or placed in service until all of the contributing drainage area has been stabilized and approved by the responsible inspector.

7.8.2 Trench Preparation

Excavate the trench to the design dimensions. Excavated materials should be placed away from the trench sides to enhance trench wall stability. Large tree roots must be trimmed flush with the trench sides in order to prevent fabric puncturing or tearing during subsequent installation procedures. The side walls of the trench should be roughened where sheared and scaled by heavy equipment.

7.8.3 Fabric Laydown

The filter fabric roll must be cut to the proper width prior to installation. This width must include sufficient material to conform to trench perimeter irregularities and for a six-inch minimum top overlap. Place the fabric roll over the trench and unroll a sufficient length to allow placement of the fabric down into the trench. Stones or other anchoring objects should be placed on the fabric at the edge of the trench to keep the lined trench open during windy periods. When overlaps are required between rolls, the upstream roll should lap a minimum of two (2) feet over the downstream roll in order to provide a shingled effect. The overlap ensures fabric continuity and ensures that the fabric conforms to the excavation surface during aggregate placement and compaction.

A partial list of suggested filter fabric brands is listed in Table 12.

Table 12 - Approved Geo-Textiles For Use in Infiltration Trenches	
Mirafi 140-N	Note: This is a partial list of acceptable filter fabrics available from suppliers for use in infiltration trenches. The use of a brand name does not constitute an endorsement by HRPDC of any particular product or company.
Supac 4NP, 4.5NP, 5NP, and 8NP	
Typar 3401	
AMOCO 4545	
EXXON Geo-textiles No. 125D, 130D, and 150D	
TerraTex SD	
Source: "Controlling Urban Runoff" - Metropolitan Washington Council of Governments.	

7.8.4 Stone Aggregate Placement and Compaction

The stone aggregate should be placed in lifts and compacted using plate compactors. As a rule of thumb, a maximum loose lift thickness of 12 inches is recommended. The compaction process ensures fabric conformity to the excavation sides, thereby reducing the potential for soil piping, fabric clogging, and settlement problems.

7.8.5 Overlapping and Covering

Following the stone aggregate placement, the filter fabric should be folded over the stone aggregate to form a six-inch (6") minimum longitudinal lap. The desired fill soil or stone aggregate should be placed over the lap at sufficient intervals to maintain the lap during subsequent backfilling.

7.8.6 Contamination

Care shall be exercised to prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate must be removed and replaced with uncontaminated stone aggregate.

7.8.7 Voids Behind Fabric

Voids should be avoided between the fabric and excavation sides. Removing boulders or other obstacles from the trench walls is one source of such voids. Natural soils should be placed in these voids at the most convenient time during construction but prior to installing fabric to ensure fabric conformity to the excavation sides. Soil piping, fabric clogging, and possible surface subsidence will be avoided by this remedial process.

7.8.8 Unstable Excavation Sides

Vertically excavated walls may be difficult to maintain in areas where the soil moisture is high or where soft cohesive or cohesionless soils predominate. These conditions may require laying back of the side slopes to maintain stability; trapezoidal rather than rectangular cross sections may result.

7.8.9 Vegetative Buffer

A vegetative buffer of at least 20 feet wide (wider if possible) should be used to intercept surface runoff from all impervious areas.

7.8.10 Traffic Control

Heavy equipment and traffic shall be restricted from travelling over the infiltration areas to minimize compaction of the soil.

7.8.11 Observation Well

An observation well, as described in subsection 7.3.8 and Figure 11 should be provided. The depth of the well at the time of installation should be clearly marked on the well cap.

8 INFILTRATION BASIN

8.1 Description

An infiltration basin is a storm runoff impoundment made by excavating a pit in, or down to, good permeable soils. The purpose of the basin is to store the storm runoff for a selected design storm or for the first flush and to slowly infiltrate it through the permeable bottom of the basin. An infiltration basin schematic is shown on Figure 13. Figure 14 shows a typical infiltration basin which receives the concentrated storm runoff. A riprap apron near the inlet is required to reduce incoming velocity. The flat basin floor with dense grass turf is used to trap sediments. An emergency spillway should be provided to bypass overflows.

Figure 15 shows an infiltration basin with a riprap pilot channel on one side of the basin which extends from the outfall of the pipe to the riser. Baseflow can flow through the pilot channel and leave the basin through a low flow pipe at the base of the riser. Any flow larger than the base flow will spread to the entire basin floor which is covered with dense grass turf. Riprap in the pilot channel should have a layer of filter fabric under it. A riser outlet can be designed to pond the runoff from a specific storm to facilitate settling of the sediments from the runoff. An emergency spillway should be provided to handle overflows.

Figure 16 shows a two-level infiltration basin and is a modification of the basin in Figure 14. The sediment forebay helps in settling heavier sediments and other objectionable materials, thus enhancing the infiltration capacity of the remaining portion of the basin.

For all designs of infiltration basins with inflow from a storm drain pipe, the hydraulic grade line of the storm drain system should be checked.

8.2 Applicability

An infiltration basin can typically be constructed for drainage areas of five (5) to fifty (50) acres. For drainage areas less than five (5) acres, it is more appropriate to use an infiltration trench or dry well. An infiltration basin has relatively large surface area requirements when compared to a trench or a dry well.

8.3 Design Criteria

8.3.1 Soil Permeability

The permeability or final infiltration rate of the underlying soil in the infiltration basin will determine how rapidly the storm runoff ponded in the basin will infiltrate into the ground. Soil textural classes with infiltration rates greater than or equal to 0.52 inches per hour should be used for the installation of the basin. This infiltration rate is associated with soil textural groups of sand, loamy sand, sandy loam, and loam.

8.3.2 Depth of Basin

A typical infiltration basin can range from three (3) feet to six (6) feet in depth. The rate and quantity of exfiltration is increased by increasing the surface area of the basin. Thus, wide and long basins with shallow depths are preferable to small and deep basins. With the passage of time, the bottom surface area of the basins will get clogged and will diminish the exfiltration rate. Excess bottom surface area can compensate for the loss of infiltration capacity.

8.3.3 Groundwater Table

The seasonal high groundwater table should be located at least two (2) to four (4) feet below the bottom of the basin. The soil permeability (infiltration rate) and the depth to the groundwater table are the two parameters which are used to determine the maximum allowable depth of the basin.

8.3.4 Proximity To Wells and Foundations

Infiltration basins should be located at least 100 feet upgradient from any drinking water supply well to minimize the possibility of groundwater contamination. Also, the basins should be located at least ten (10) feet down-gradient and 100 feet up-gradient from building foundations.

8.3.5 Basin Slopes

The main objective in the infiltration basin design is to achieve a uniform ponding depth over the entire surface area of the basin. The uniform ponding depth can be achieved by grading the floor of the basin as close to a zero slope as possible. Any low spots should be avoided to prevent ponding of the surface runoff. Deposition of solids in low areas may cause clogging of the underlying soil.

The storm runoff entering the basin should be spread out evenly over the bottom surface of the basin to promote better infiltration. This can be achieved by providing an apron or a level spreader.

The side slopes of the basin should be three to one (3:1) or flatter to help in establishing proper vegetation and grass cover. Water tolerant grass should be planted on the bottom and sides of the basin. A dense growth of grass will help to prevent scouring of the basin floor and sides.

8.3.6 Design Storm

Infiltration Basins can be designed for a specific storm or for the first flush runoff volume. If for first flush, the basin storage volume can be based on 0.5 inches of runoff per impervious acre in the contributing site area.

Larger and less frequent storms will bypass the basin. Additional storage will be needed to provide control of storm runoff from these larger storms. This can be achieved by providing a conventional riser pipe in the basin.

Concentrated flows with erosive velocities should be prevented from entering the basin. An emergency spillway should be provided for all basins created by an embankment. The emergency spillway design should comply with state and local requirements.

8.3.7 Storage Time/Maximum Draining Time

All infiltration basins should be designed to drain within a maximum time of three (3) days (72 hours), or a minimum time of two (2) days (48 hours). These values are derived from literature. The Virginia Stormwater Management Regulations recommend two (2) days (48 hours).

8.3.8 Runoff Filtering

Sediments in the storm runoff and objectionable floating materials should be prevented from entering the basin. The longevity of the basin can be increased by installing sediment forebays near the inlets to trap incoming sediment loads. It is recommended to provide a minimum of 20 feet vegetative buffer around the basin to prevent sediments from entering the basin.

8.4 Design Examples

Design an infiltration basin for a residential development of 5.2 acres. Soil borings indicate that the soil is sandy loam with an infiltration rate of 1.02 inches per hour. The increase in runoff depth is 0.30 inch for a one-inch (1") rainfall. The depth of seasonal ground water table is determined to be five (5) feet. Side slopes of the basins should be three to one (3:1) and top width of the basin should be 40 feet.

DESIGN OF INFILTRATION BASIN FOR 1" RAINFALL

Project: 8.4 INFILTRATION BASIN FOR 5.2 ACRE RESIDENTIAL SITE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 1.02

Maximum Allowable Ponding Time of Basin(Hrs): 72

Depth to Seasonal High Groundwater Table(Ft): 5.00

Min. Dist. from Basin bottom to Groundwater Table(Ft): 2.00

***** Maximum Basin Depth(Ft): 3.00 *****

----->>> Design Input Parameters <<<-----

Increase in Runoff Depth(In): 0.30

Contributing Drainage Area(Sq Ft): 226512

Depth of Basin(Ft): 3.00

Amount of Rainfall(In): 1.00

Side Slope of Basin(Ratio): 3.0

Top Width of Basin(Ft): 40

***** Basin Length(Ft): 70 *****

Example 2

Design an infiltration basin for a commercial site of 5.0 acres with an increase in runoff depth of 0.50 inch associated with a rainfall of 1.3 inches. Soil borings indicate that the soil is sandy loam with an infiltration rate of 0.9 inches per hour. The depth of the seasonal ground water table is determined to be 4.5 feet. Side slopes of the basin should be three to one (3:1) and top width of the basin should be 35 feet.

DESIGN OF INFILTRATION BASIN FOR 1.3" RAINFALL

Project: 8.4 INFILTRATION BASIN FOR 5.0 ACRES COMMERCIAL SITE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.90

Maximum Allowable Ponding Time of Basin(Hrs): 72

Depth to Seasonal High Groundwater Table(Ft): 4.50

Min. Dist. from Basin bottom to Groundwater Table(Ft): 2.00

******* Maximum Basin Depth(Ft): 2.50 *******

----->>> Design Input Parameters <<<-----

Increase in Runoff Depth(In): 0.50

Contributing Drainage Area(Sq Ft): 217800

Depth of Basin(Ft): 2.50

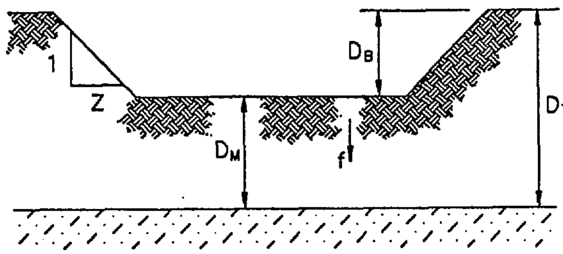
Amount of Rainfall(In): 1.30

Side Slope of Basin(Ratio): 3.0

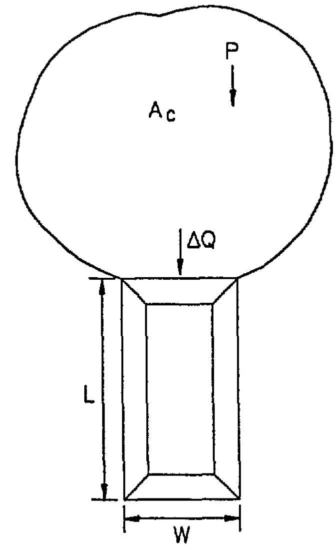
Top Width of Basin(Ft): 35

******* Basin Length(Ft): 145 *******

SIDE VIEW



TOP VIEW



f = Infiltration Rate(In/Hr)

T_P = Maximum Allowable Ponding Time(Hrs)

D_T = Depth to Seasonal High Groundwater Table(Ft)

D_M = Min Dist from Basin bottom to Groundwater Table(Ft)

ΔQ = Increase in Runoff Depth(In)

A_C = Contributing Drainage Area(Sq Ft)

D_B = Depth of Basin(Ft)

P = Amount of Rainfall(In)

Z = Side Slope Ratio (N:1)

W = Top Width of Basin (Ft)

L = Top Length of Basin (Ft)

$$L = \frac{\frac{\Delta Q}{12} A_C + Z D_B^2 (W - 2 Z D_B)}{W(D_B - \frac{P}{12}) - Z D_B^2}$$

INFILTRATION BASIN SCHEMATIC

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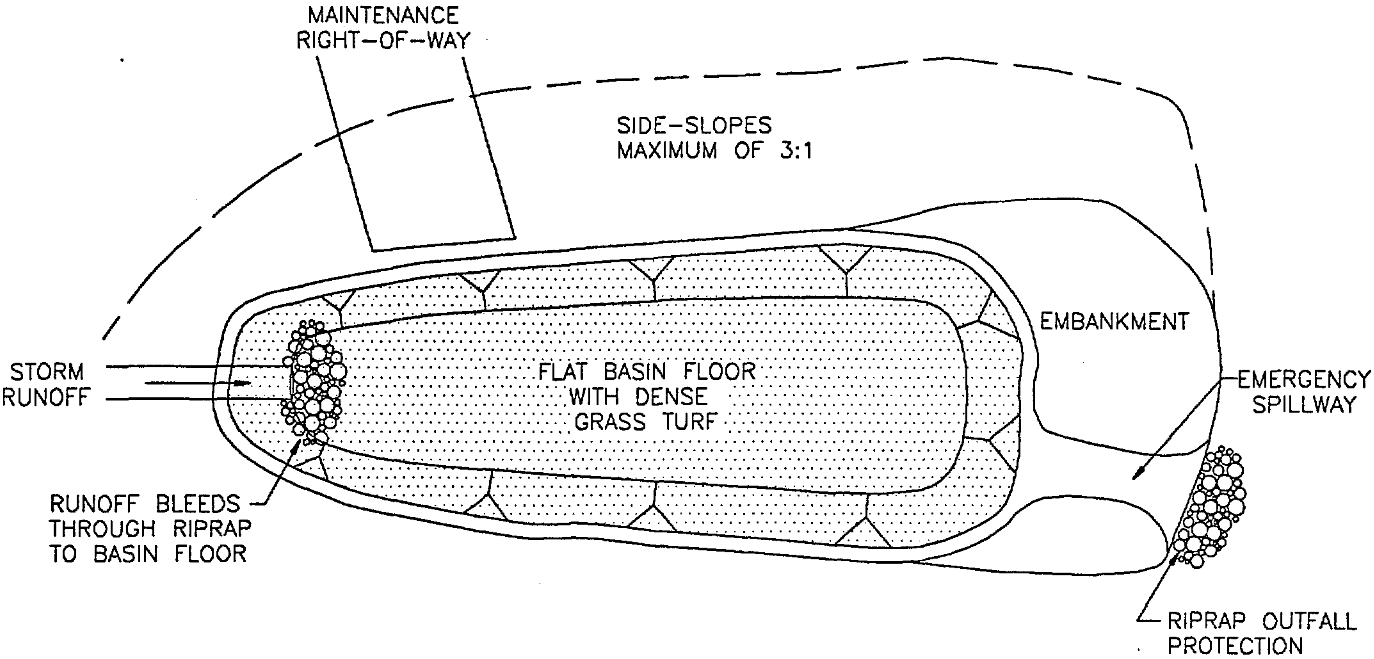
HAMPTON ROADS
PLANNING DISTRICT COMMISSION

REVS'D:
CHK'D:
DATE:

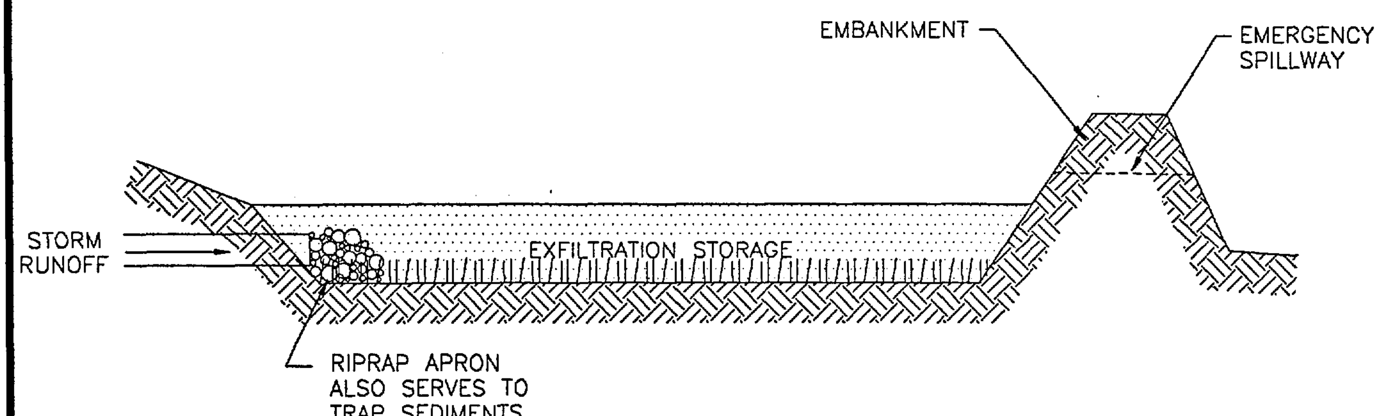
REVS'D:
CHK'D:
DATE:

DRAWN: K.F.F.
CHK'D: V.P.M
DATE: OCT. 1990
SCALE: N.T.S.

FIGURE 14



TOP VIEW



SIDE VIEW

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DATE: OCT. 1990
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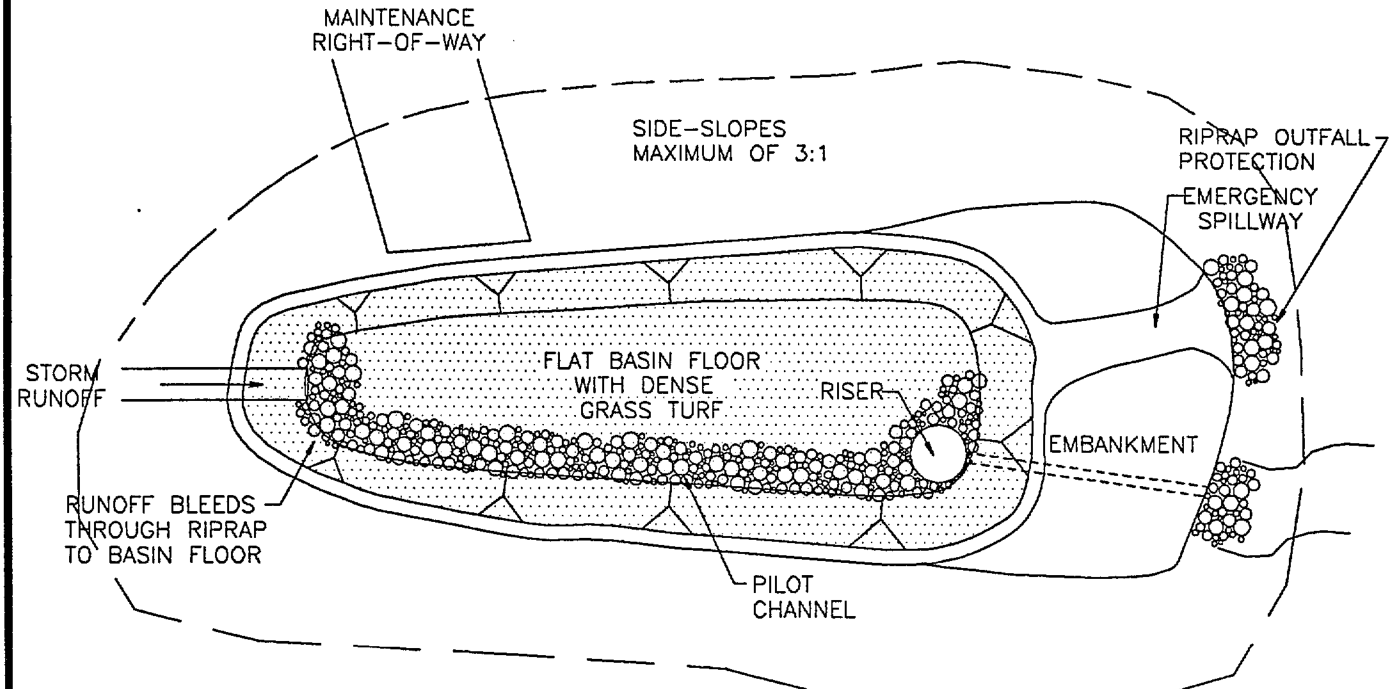
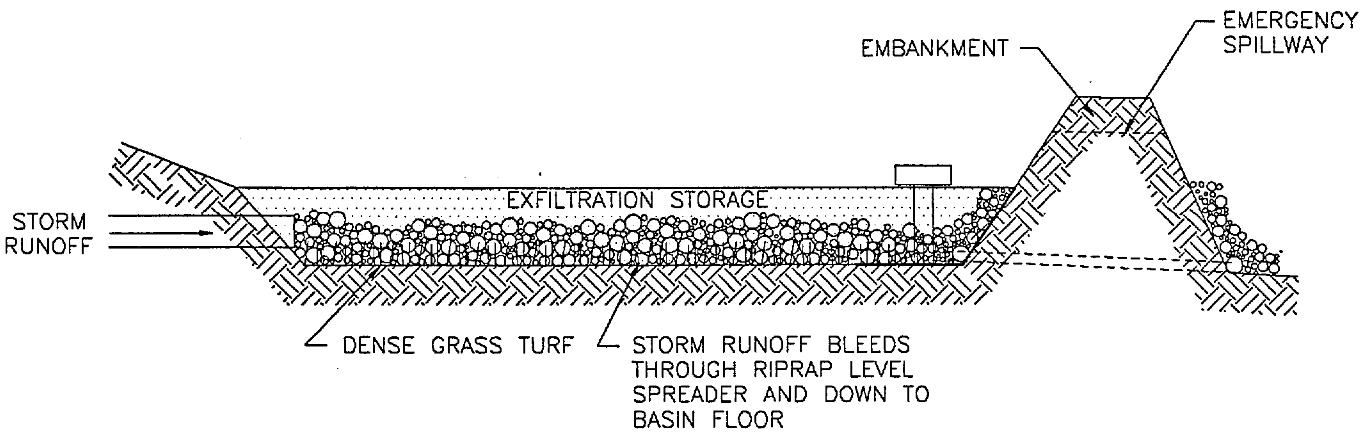
FIGURE 15



TOP VIEW



SIDE VIEW

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CHK'D:
DATE:

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DATE: OCT. 1990
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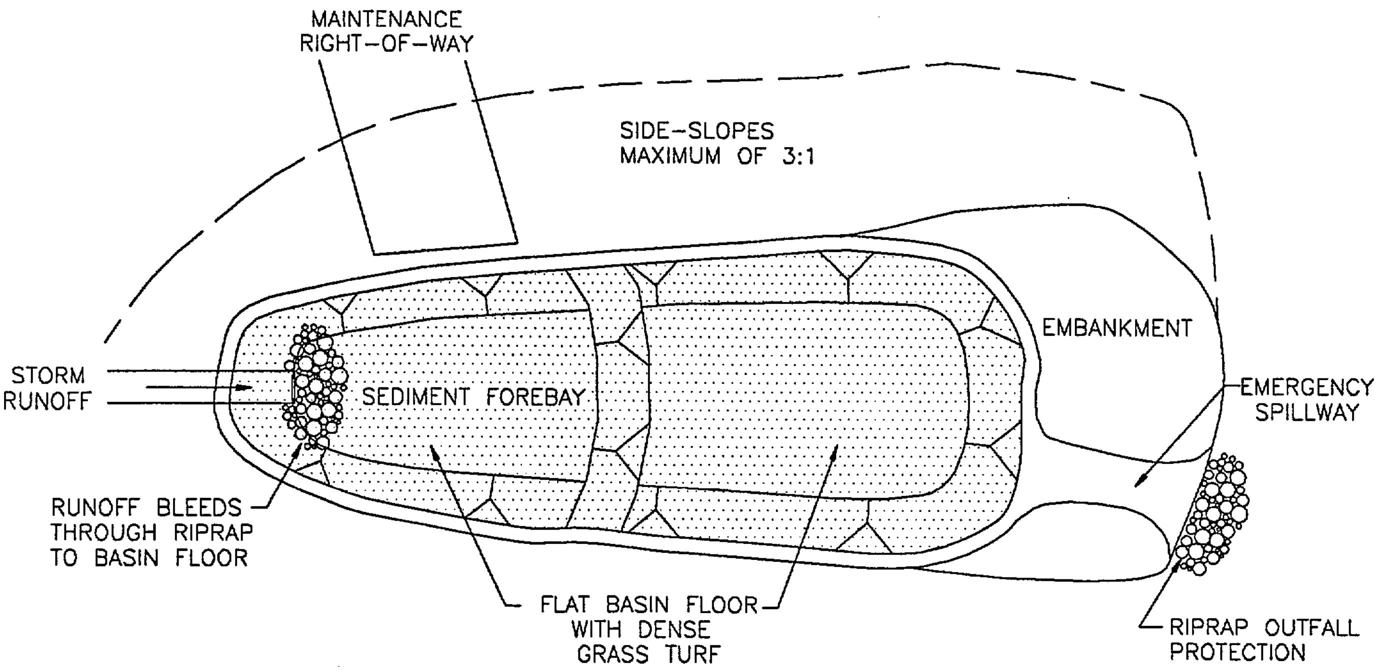
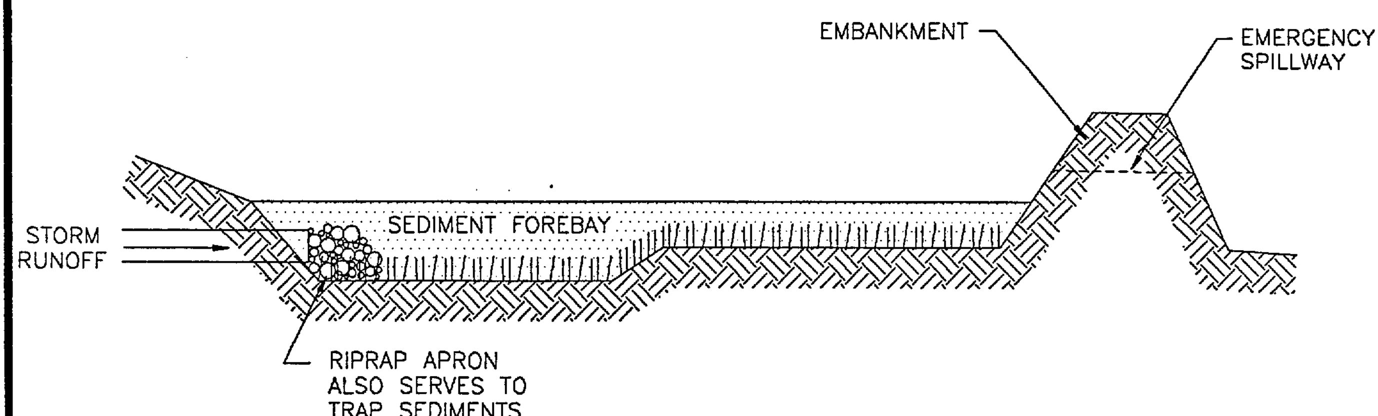
FIGURE 16



TOP VIEW



SIDE VIEW

TWO-LEVEL INFILTRATION BASIN

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DATE:

REVS'D:
CHK'D:
DATE:

DRAWN: R.E.C.
CHK'D: V.P.M.
DATE: OCT. 1990
SCALE: N.T.S.

8.5 Maintenance Requirements

8.5.1 Inspection

The infiltration basin initially should be inspected after every major storm to determine how long the storm runoff remains in the basin. Standing water in the basin for more than 72 hours after a storm indicates problems with infiltration. Semi-annual or annual inspections should be performed to determine proper functioning of the basin.

8.5.2 Mowing

The basin floor and sides should be mowed as needed to prevent unsightly growth of grass or weeds. Mowing operations should be performed during dry periods because at other times the bottom of the basin may be soggy. If the infiltration basin is being used as a passive recreational area, more frequent mowing may be needed.

8.5.3 Erosion Control

Eroding or barren areas at the bottom of the basin or on the sides should be immediately re-vegetated.

During the regular semiannual or annual inspection, other items like condition of the embankment, riprap at the inlet, outlet, and pilot channel (if used) should be checked. Trash should be removed from basins with no outlets.

8.5.4 Non-Routine Maintenance

Infiltration basins with risers and outlet pipes may not need replacement for a long time depending on the type of materials used for their construction. Concrete outlet pipes may last for 40-50 years and corrugated metal outlet pipe longevity may be about the same.

The life expectancy of the basin depends on the amount of sediment being trapped and the infiltration capacity of the underlying soil. The literature suggests that a well-maintained basin may remain functional for five (5) to ten (10) years before the trapped sediments need to be removed. Sediment removal should be undertaken when the bottom of the basin is dry. Trapped sediments can be used as a fill material for new sites or used as spreading soil in gardens. Sediments should be removed with light equipment. The bottom of the basin should be scarified to restore its infiltration rate and re-vegetated.

8.5.5 Total Maintenance Costs

Based on current experience with infiltration basins in other parts of the country, it is reasonable to assume the total annual maintenance costs to be three (3) to five (5) percent of the initial construction cost.

8.6 Life Expectancy

The main cause of failure of an infiltration basin as a BMP is clogging of the basin bottom with sediments from the storm runoff. Infiltration basins should be constructed after the contributing area serving the basin has been stabilized. Based on experience in the State of Maryland, infiltration basins may typically function properly anywhere from six (6) months to two (2) years. Life expectancy may also depend on the land use in the contributing area and the contents of the storm runoff. Proper construction inspection and maintenance may enhance the useful life of the infiltration basin as a BMP.

8.7 Cost

A general planning estimate of infiltration basin costs can be made by using the following equation:

$$C = 13.2\, V_s^{0.69}$$

where C = construction cost in 1990 dollars

V_s = basin storage volume in cubic feet.

The above equation to estimate basin construction costs does not include land acquisition costs or the cost of any pretreatment structures. An additional 25% should be added to the construction cost to cover contingency costs.

8.8 Construction Specifications

As with other infiltration practices, infiltration basins, if not constructed properly may have a high rate of failure. The most common cause of failure is poor sediment control on upland contributing areas and incorrect location of the basin in soils with poor infiltration rates. The infiltration basin should be constructed after the contributing area has been stabilized.

8.8.1 Excavation

Compaction of underlying soils by heavy equipment should be prevented. After final grading of the basin bottom is completed, the bottom should be scarified to provide a porous surface texture for better infiltration rates.

8.8.2 Lining Material

Infiltration basins should be lined with a 6-inch to 12-inch layer of filter material such as coarse sand to help prevent the buildup of impervious deposits on the soil surface. The filter layer should be replaced or cleaned when it becomes clogged.

Establishing dense vegetation on the basin side slopes and floor is recommended. A dense vegetative stand will not only prevent erosion and sloughing (side slope erosion), but will also provide a natural means of maintaining relatively high infiltration rates. Erosion protection for inflow points to the basin should also be provided.

The basin should be stabilized with vegetation within one (1) week after construction.

9 UNDERGROUND STORAGE TRENCH

9.1 Description

An underground storage BMP functions like an infiltration trench, except that it can accept concentrated runoff. The concentrated runoff should be pretreated before it enters the underground storage trench. In some situations where there is good permeable underlying soil and there is not enough space available to install an infiltration trench, an underground storage trench can be constructed under paved areas. This is not a recommended practice for all sites, because it is extremely costly; it is hard to replace the trench if the pavement fails; and, maintenance of the trench is a problem. An underground storage trench can be installed under a grass lined swale to augment infiltration of runoff. A schematic of an underground storage trench is shown on Figure 17. Detail of an underground storage trench is shown on Figure 18.

9.2 Applicability

As mentioned above, the application of an underground storage trench is similar to an infiltration trench. Underground storage trenches are primarily on-site control BMPs and are generally applicable to small drainage areas up to five (5) acres. This BMP can be installed in residential developments or commercial areas.

9.3 Design Criteria

9.3.1 Soil Permeability

Soil textural classes with infiltration rates greater than or equal to 0.27 inches per hour should be used for the installation of underground storage trenches. This infiltration rate is associated with soil textural groups of sand, loamy sand, sandy loam, loam, and silt loam. The infiltration rate of the underlying soil and the depth of the groundwater table are the major limiting factors in the selection and feasibility of the underground storage trench as a BMP.

9.3.2 Depth of Trench

The final infiltration rate of the soil below the underground storage trench determines the maximum allowable trench depth. A trench with a grass covered surface should have at least one (1) foot of overlying soil above the stone aggregate reservoir. The surface area of the trench can be minimized by making the trench as deep as feasible. The trench can also be made shallow and broad. The increased surface area of the bottom of the trench increases exfiltration rates and provides more area for soil filtering of pollutants. A larger trench bottom also helps in reducing clogging at the soil/filter cloth interface by providing exfiltration over a wide area.

9.3.3 Groundwater Table

The seasonal high groundwater table should be located at least two (2) to four (4) feet below the bottom of the trench. The soil permeability (infiltration rate) and the groundwater table are the two parameters which determine the maximum allowable depth of the trench.

9.3.4 Proximity To Wells and Foundations

Underground storage trenches should be located at least 100 feet upgradient from any drinking water supply well to minimize the possibility of groundwater contamination. Also the trenches should be located at least ten (10) feet downgradient and 100 feet upgradient from building foundations.

9.3.5 Design Storm

Underground storage trenches can be designed for a specific storm or for the first flush runoff volume. If for first flush, the trench storage volume can be sized based on 0.5 inches of runoff per impervious acre in the contributing site area.

Underground storage trenches are normally designed for water quality. As such, a significant portion of the runoff volume (storms producing more than 0.5 inches of runoff) will bypass the trench and is not infiltrated.

9.3.6 Storage Time/Maximum Draining Time

All underground storage trenches should be designed to drain within a maximum time of three (3) days (72 hours), or a minimum time of two (2) days (48 hours). These values are derived from literature. The Virginia Stormwater Management Regulations recommend two (2) days (48 hours).

9.3.7 Stone Aggregate

The stone aggregate which fills the underground storage trench forms the reservoir through which the storm runoff passes and is filtered. The aggregate material should be clean, washed stone. Wash run gravel is preferred. The City of Virginia Beach recommends using James River Stone as aggregate. The clean washed stone aggregate should have a maximum diameter of three (3) inches and a minimum diameter of one (1) inch. Void spaces for the stone aggregate are normally within the range of 30 to 40 percent. A table showing open graded coarse aggregates is included in the Appendix.

9.3.8 Observation Well

An observation well shall be installed in the underground storage trench. The observation well should be a perforated PVC pipe, four (4) to six (6) inches in diameter. The pipe should be located in the center of the trench and the bottom should rest on a plate. The top of the well should be capped to prevent vandalism and tampering.

The observation well helps in monitoring the function of the trench. The water level in the trench should be measured after a storm event. If the trench does not completely drain after three (3) days, it indicates that the trench is not functioning properly and remedial steps may need to be taken to improve the performance.

9.3.9 Runoff Filtering

It is important to prevent any floatable material, settleable solids, grease, and oil from entering the underground storage trench. Runoff filtering devices such as water quality inlets can be used in front of the trench to prevent objectionable materials from entering the trench. All trenches with surface inlets shall be designed to capture the sediments and other material before the storm runoff discharges into the stone aggregate reservoir.

The sides of the trench should be lined with filter fabric to prevent the entry of sediments into the trench. The bottom of the trench, if constructed in good permeable soil, can be lined with a six-inch (6") layer of sand or filter fabric.

9.3.10 Overflow Requirements

In all cases, the overflow path of storm runoff exceeding the capacity of the trench should be evaluated and accommodated. The trenches are designed to treat the first flush volume of runoff and control small drainage areas.

9.4 Design Examples

Design an underground storage trench for a one (1) acre commercial site with a paved area of 20,000 square feet generating 0.4 inches of runoff for a one-inch (1") rainfall. The runoff should be pretreated before entering the underground storage trench. Underlying soil is silt loam with an infiltration rate of 0.27 inches per hour. Groundwater table is at ten (10) feet from the ground. The trench will be installed under a grassed area one (1) foot deep. Depth of the trench should not be more than three (3) feet.

DESIGN OF UNDERGROUND STORAGE FOR 1" RAINFALL

Project: 9.4 DESIGN FOR UNDERGROUND TRENCH FOR ONE(1) AC. SITE

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.27

Maximum Allowable Storage Time of Trench(Hrs): 72

Void Ratio Trench Medium: 0.4

Depth to Seasonal High Groundwater Table(Ft): 10.0

Min. Dist. from Trench bottom to Groundwater Table(Ft): 2.0

***** Maximum Trench Depth(Ft): 4.0 *****

----->>> Design Input Parameters <<<-----

Increase in Runoff Depth(In): 0.40

Contributing Drainage Area(Sq Ft): 20000

Depth of Trench (Ft): 3.0

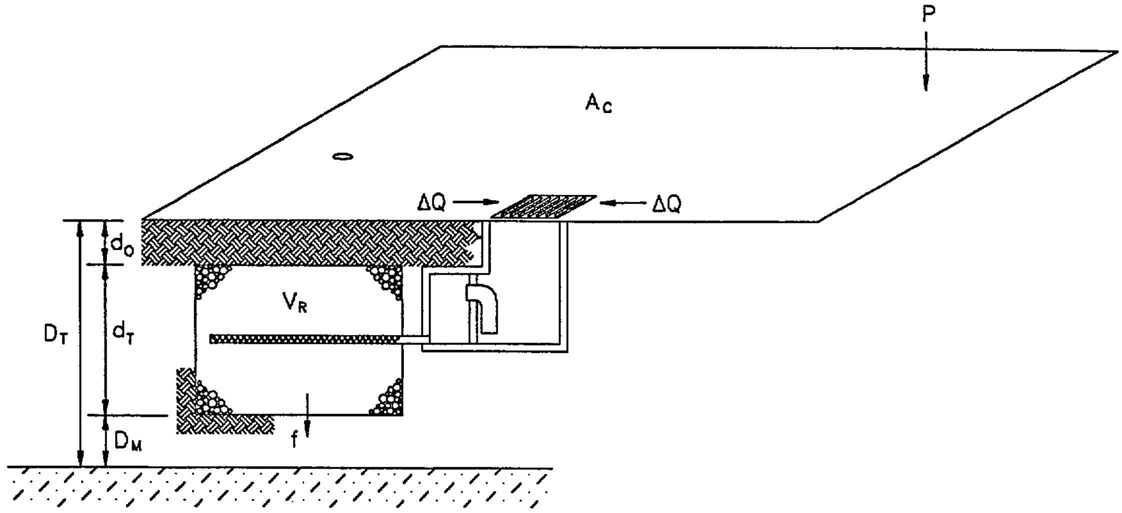
Depth over Trench(Ft): 1.0

Amount of Rainfall(In): 1.00

Trench Filling Time(Hrs): 2

***** Trench Area(Sq Ft): 574 *****

FIGURE 17



f = Infiltration Rate(In/Hr)

T_S = Maximum Allowable Storage Time of Trench(Hrs)

V_R = Void Ratio Trench Medium

D_T = Depth to Seasonal High Groundwater Table(Ft)

D_M = Min Dist from Trench bottom to Groundwater Table(Ft)

A_C = Contributing Drainage Area(Sq Ft)

ΔQ = Increase in Runoff Depth(In)

d_T = Depth of Trench(Ft)

d_O = Depth over Trench(Ft)

P = Amount of Rainfall(In)

T = Trench Filling Time(Hrs)

For First Flush Design:

$$\text{Area of Trench} = \frac{\frac{\Delta Q}{12} A_C}{V_R\, d_T}$$

$$\text{Area of Trench} = \frac{\frac{\Delta Q}{12} A_C}{V_R\, d_T - \frac{P}{12} + \frac{f}{12} T}$$

UNDERGROUND STORAGE SCHEMATIC

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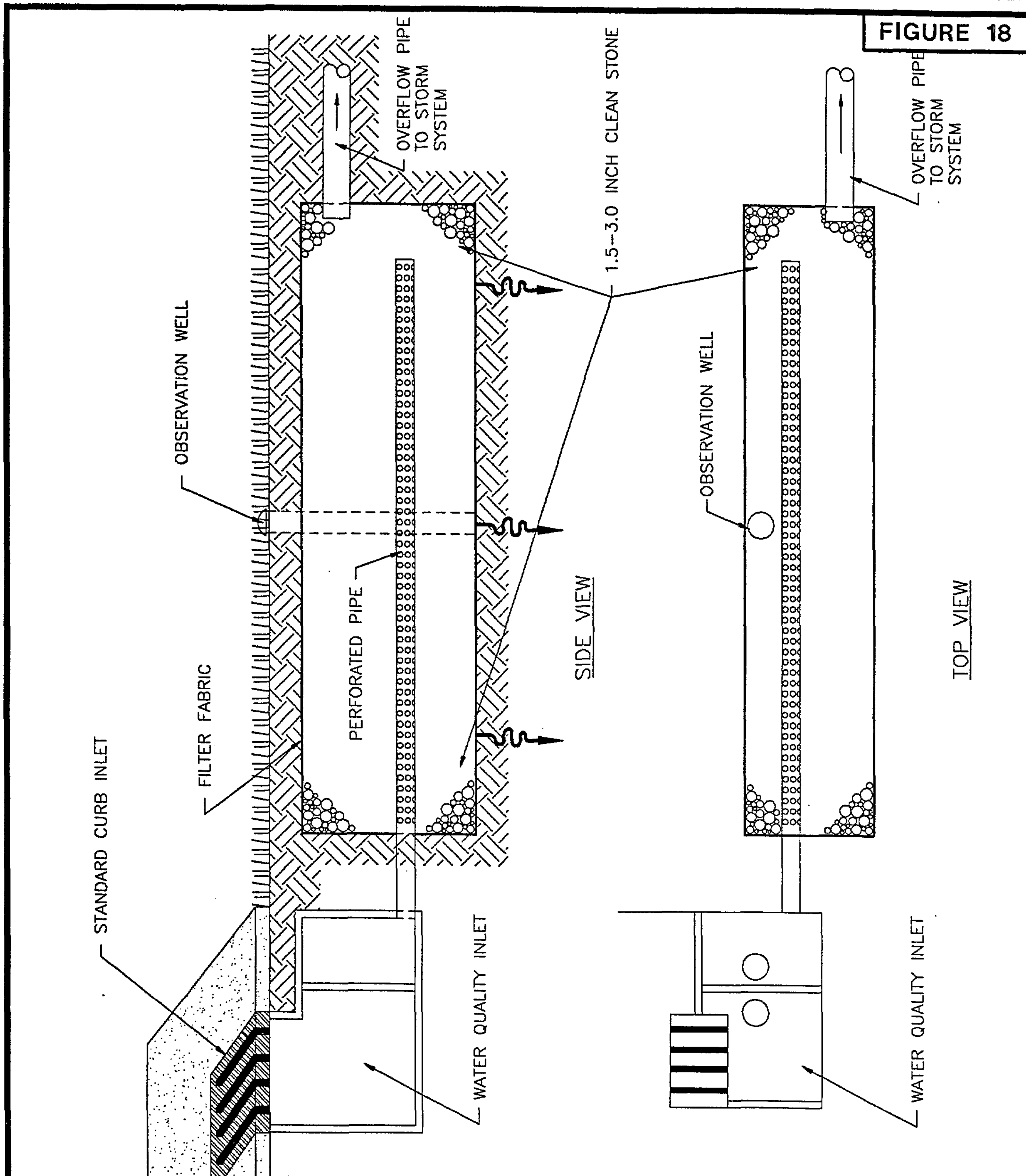
FIGURE 18



UNDERGROUND STORAGE DETAIL

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9.5 Maintenance Requirements

An underground storage BMP may require more inspection visits and need more enforcement than an infiltration trench. The water quality inlet should be inspected at least twice a year to ensure proper functioning. If needed, contents of the chambers should be pumped out and properly disposed of at least twice a year.

Rehabilitation of underground storage trenches may cost the same as the initial construction cost. An annual set-aside of 10-15% of the initial construction cost for underground storage trenches may be required to cover routine/non-routine maintenance expenditures. These estimates are based on existing information and may vary from site to site and differ for each jurisdiction. Reliable maintenance costs and life expectancy of underground storage trenches will become more accurate with experience and time.

9.6 Life Expectancy

Enough information on the useful life expectancy of underground storage trench is not available. A rough estimate of the life expectancy of an underground storage trench is six (6) months to two (2) years. Proper construction inspection, and maintenance may enhance useful life expectancy.

9.7 Cost

The cost of installing an underground storage trench could be 30 to 40 percent more than an infiltration trench because a pretreatment facility should be constructed before this BMP. An underground storage trench may range from $5,000 to $15,000.

9.8 Construction Specifications

The specifications listed for infiltration trenches are applicable for underground storage trenches. They are listed here for ease of use.

9.8.1 Timing

An underground storage trench should not be constructed or placed in service until all of the contributing drainage area has been stabilized and approved by the responsible inspector.

9.8.2 Trench Preparation

Excavate the trench to the design dimensions. Excavated materials should be placed away from the trench sides to enhance trench wall stability. Large tree roots must be trimmed flush with the trench sides in order to prevent fabric puncturing or tearing during subsequent installation procedures. The side walls of the trench should be roughened where sheared and scaled by heavy equipment.

9.8.3 Fabric Laydown

The filter fabric roll must be cut to the proper width prior to installation. This width must include sufficient material to conform to trench perimeter irregularities and for a six-inch minimum top overlap. Place the fabric roll over the trench and unroll a sufficient length to allow placement of the fabric down into the trench. Stones or other anchoring objects should be placed on the fabric at the edge of the trench to keep the lined trench open during windy periods. When overlaps are required between rolls, the upstream roll should lap a minimum of two (2) feet over the downstream roll in order to provide a shingled effect. The overlap ensures fabric continuity and ensures that the fabric conforms to the excavation surface during aggregate placement and compaction.

A partial list of suggested filter fabric brands is given below:

Table 13 - Approved Geo-Textiles For Use in Underground Storage Trenches	
Mirafi 140-N	Note: This is a partial list of acceptable filter fabrics available from suppliers for use in underground storage trenches. The use of a brand name does not constitute an endorsement by HRPDC of any particular product or company.
Supac 4NP, 4.5NP, 5NP, and 8NP	
Typar 3401	
AMOCO 4545	
EXXON Geo-textiles No. 125D, 130D, and 150D	
TerraTex SD	
Source: "Controlling Urban Runoff" - Metropolitan Washington Council of Governments.	

9.8.4 <u>Stone Aggregate Placement and Compaction</u>

The stone aggregate should be placed in lifts and compacted using plate compactors. As a rule of thumb, a maximum loose lift thickness of 12 inches is recommended. The compaction process ensures fabric conformity to the excavation sides, thereby reducing the potential for soil piping, fabric clogging, and settlement problems.

9.8.5 <u>Overlapping and Covering</u>

Following the stone aggregate placement, the filter fabric should be folded over the stone aggregate to form a six-inch (6") minimum longitudinal lap. The desired fill soil or stone aggregate should be placed over the lap at sufficient intervals to maintain the lap during subsequent backfilling.

9.8.6 <u>Contamination</u>

Care shall be exercised to prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate must be removed and replaced with uncontaminated stone aggregate.

9.8.7 Voids Behind Fabric

Voids should be avoided between the fabric and excavation sides. Removing boulders or other obstacles from the trench walls is one source of such voids. Natural soils should be placed in these voids at the most convenient time during construction but prior to installing fabric to ensure fabric conformity to the excavation sides. Soil piping, fabric clogging, and possible surface subsidence will be avoided by this remedial process.

9.8.8 Unstable Excavation Sides

Vertically excavated walls may be difficult to maintain in areas where the soil moisture is high or where soft cohesive or cohesionless soils predominate. These conditions may require laying back of the side slopes to maintain stability; trapezoidal rather than rectangular cross sections may result.

9.8.9 Vegetative Buffer

A vegetative buffer of at least 20 feet wide (wider if possible) should be used to intercept surface runoff from all impervious areas.

9.8.10 Traffic Control

Heavy equipment and traffic shall be restricted from travelling over the infiltration areas to minimize compaction of the soil.

9.8.11 Observation Well

An observation well, as described in subsection 9.3.8 and Figure 18 should be provided. The depth of the well at the time of installation should be clearly marked on the well cap.

10 POROUS PAVEMENT

10.1 Description

Porous pavement is comprised of porous asphalt material and a high void aggregate base that temporarily stores the storm runoff and the rain falling onto the paved asphalt surface. The stored runoff in the aggregate base is then infiltrated into the permeable underlying soil. In most cases, porous pavement consists of four (4) layers as described below (from bottom to top):

- Minimally compacted subbase consisting of undisturbed existing soil. Two-inch (2") thick layer of 0.5 inch diameter gravel on top of a filter fabric layer. This layer acts as a filter course.
- Reservoir base course consisting of 1.5 - 3.0 inch diameter stone aggregate (aggregate subbase). The thickness of this layer is determined from the runoff volume that needs to be stored. This course acts as a stone reservoir.
- Two-inch (2") thick layer of 0.5-inch diameter gravel to stabilize the reservoir base course. This layer acts as a filter course.
- Porous asphalt pavement course with a thickness that is based on bearing strength and pavement design requirements. In most applications 2.5 - 4.0 inches thick is found to be sufficient.

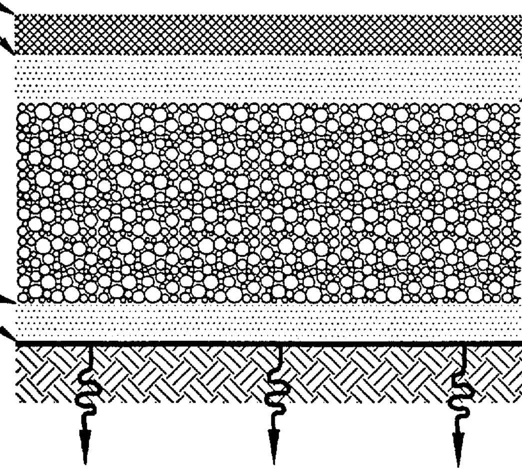
Based on the infiltration capacity of the underlying soil, auxiliary drainage structures like pipe drains, french drains, etc. may also need to be installed at the bottom of the porous pavement.

The storm runoff infiltrates through the pores of the porous asphalt pavement course into the void spaces in the underground aggregate base course. Runoff then exfiltrates out of the reservoir base course into the underlying subbase. For underlying subsoils with marginal or low infiltration rates, the exfiltrated runoff may be collected by underdrain pipes and routed to an outfall structure.

Porous pavements should be restricted to sites with contributing drainage areas of 1/4 to ten (10) acres.

A detail of typical porous pavement illustrating the four (4) layers is shown on Figure 19. A porous pavement schematic is shown on Figure 20.

FIGURE 19



POROUS PAVEMENT DETAIL

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10.2 Applicability

The use of porous pavement as a BMP is restricted to low traffic volume parking areas. It is a substitute for conventional asphalt pavement. It is feasible on sites with gentle to flat slopes and subsoils with moderate permeability. Possible areas for use of this BMP include:

- Overflow parking areas.
- Emergency stopping areas, parking lanes, and cross-over lanes on divided highways.
- Low traffic volume roads
- Driveways attached to residential lots.

10.3 Design Criteria

10.3.1 Soil Permeability

The permeability or infiltration rate of the underlying soil will determine the depth of the aggregate subbase of the porous pavement. Soil textural classes with infiltration rates greater than or equal to 0.52 inches per hour should be used for the design of the porous pavement. This infiltration rate is associated with soil textural groups of sand, loamy sand, sandy loam, and loam. A thorough examination of the permeability of the underlying soils is the key element in the design of the porous pavement. At least two (2) test soil borings should be taken to determine the permeability of the underlying soil.

10.3.2 Groundwater Table

The seasonal high groundwater table should be located at least two (2) to four (4) feet below the bottom of the reservoir base course (aggregate subbase).

10.3.3 Depth of Aggregate Base Course

A typical porous pavement reservoir base course can range from two (2) to four (4) feet. A shallow reservoir base course with a large bottom surface area is preferred over a deep and smaller bottom surface area base course.

10.3.4 Proximity of Wells and Foundations

Porous pavement should be located at least 100 feet upgradient from a drinking water supply well and should be at least ten (10) feet downgradient from building foundations.

10.3.5 Pavement Slope

Porous pavement should be used on sites with slopes less than five (5) percent.

10.3.6 Storage Time/Maximum Draining Time

The reservoir base course should be designed to completely drain within a maximum of three (3) days (72 hours), and a minimum of two (2) days (48 hours).

10.3.7 Frost Heave

If a soil with a high susceptibility to frost heaving is being considered (for example silt loam), the reservoir base course should extend below the frost line to allow for proper drainage. In most cases, this depth below the frost line will exceed the depth of storage required to control the runoff volume from the site.

10.3.8 Pavement Design

The traffic intensity over the porous pavement is the key factor in determining the thickness of the porous asphalt pavement course. The traffic intensity is defined by the average daily Equivalent Axle Load (EAL), which is based on the equivalent of 18,000 pounds (18 Kips) axle load in the design lane.

10.3.9 Calculation of Void Space

Void space should be calculated according to the testing procedure recommended in Federal Highway Administration Report No. FHWA-RD-74-2. Design of Open-Graded Asphalt Friction Courses (Smith, Rice, and Spelman, 1974). The volume of the sample should be measured mechanically rather than calculated from a water displacement method because a great deal of water is absorbed.

10.3.10 Aggregate Gradation

The gradation required to obtain a porous asphaltic pavement is of the "open" graded type as contrasted to the "dense" graded type which is capable of close packing. The following aggregate specification is recommended.

Table 14 - Aggregate Gradation for Porous Pavement

U.S. Sieve Series Size	Opening (mm)	Specification: Percent Passing by Weight
1/2-inch	12.70	100
3/8-inch	9.51	95-100
#4	4.76	30-50
#8	2.38	5-15
#200*	0.074	2-5

* Aggregate should be uniformly graded between the #8 and #200 sieve.
Source: Maryland Standard and Specification for Infiltration Practices.

Open graded mixes, due to their relatively high permeability to air and water, provide good resistance and durability to freeze/thaw conditions and to asphalt film oxidation.

10.3.11 Type and Quality of Aggregate

The aggregates selected for porous pavement construction should meet requirements of the standard specification for "Crushed Stone, Crushed Slag, and Crushed Gravel for Dry- or Water-Bound Macadam Base and Surface Courses of Pavements," ASTM D 693-77, with two (2) exceptions. First, the gradation test must be of the open graded type described here. Second, a soundness test is required, as specified in ASTM D 692-79, "Coarse Aggregate for Bituminous Paving Mixtures," to determine if the aggregate is susceptible to disintegration by water.

10.3.12 Asphalt Cement Grade in Mix

The suggested viscosity grade of asphalt cement to be used is AC-20 of AASHTO M-226-73 I. This grade is to be considered a tentative starting point because test results obtained from the design process may indicate an advantage or a necessity to alter the asphalt grade.

10.3.13 Mixing Temperatures

To ensure that the individual aggregate particles are completely surrounded by asphalt, and that the asphalt is tightly bound to each particle, the temperature of mixing at the hot mix plant shall be rigidly controlled. Too low a mixing temperature will result in inadequate asphalt binding and coverage of the aggregate, while too high a mixing temperature will allow asphalt to drain from the mix, resulting in a lower asphalt content and decreased strength. Suitable mixing temperatures range from 230 to 260 degrees Fahrenheit, but the lower end of that range (230° to 240°F) is recommended.

10.3.14 Asphalt Content in Mix

For road paving durability and to prevent too rapid hardening of the asphalt, it is desirable to have the highest asphalt content possible in the mix. Too much asphalt would separate out under traffic, so that maximum asphalt content is generally limited by that factor. Experience has shown that 5.5 percent by weight is the minimum recommended asphalt content. Asphalt content should be determined according to the testing procedure recommended in Federal Highway Administration Report No. FHWA-RD-74-2, already cited. The Marshall Design method for determining mix content is not recommended. Using a 5.5 percent asphalt content and the recommended six-inch (6") minimum surface course, a 0.6-inch rainfall reservoir capacity is obtained with an infiltration rate of 176 inches per hour. A four-inch (4") minimum surface course is recommended by the Asphalt Institute.

10.3.15 Traffic Control

Experience has shown the need for close control of contractor vehicles on newly installed areas of porous asphalt pavement. Damage to pavement porosity results chiefly from abuse during the early life of the pavement. Normally, paving is done while heavy construction or earth moving is continuing in an area. The pavement is thus subjected to mud and dirt from contractor vehicles for up to several months, and the continual passage of these vehicles compacts the dirt into the pores. Only if caked mud is cleaned from vehicle wheels and the pavement is cleaned daily by sweeping and high-pressure water washing can porosity be retained. Clogging can be further minimized by proper use of curbing to prevent surrounding soils from washing onto the pavement surface.

10.4 Design Examples

Design porous pavement for one (1) acre of parking lot to control runoff of 0.8 inch contributed by a one-inch (1") rainfall. Underlying soils have been tested at an infiltration rate of 0.58 inches per hour. Depth to groundwater table is eight (8) feet. Depth of porous pavement should be two (2) feet. Design the pavement for first flush also.

DESIGN OF POROUS PAVEMENT FOR 1" RAINFALL

Project: 10.4 DESIGN EXAMPLE FOR ONE (1) ACRE PARKING LOT

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.58

Max. Allowable Storage Time of Aggregate Subbase(Hrs): 72

Void Ratio Aggregate Subbase: 0.4

Depth to Seasonal High Groundwater Table(Ft): 8.0

Min. Dist. from Aggr. Subbase to Groundwater Table(Ft): 2.0

******* Maximum Subbase Depth(Ft): 5.5 *******

----->>> Design Input Parameters <<<-----

Increase in Runoff Depth(In): 0.80

Contributing Drainage Area(Sq Ft): 43560

Depth of Aggregate Subbase(Ft): 2.0

Amount of Rainfall(In): 1.00

Aggr. Subbase Reservoir Filling Time(Hrs): 2

******* Porous Pavement Area(Sq Ft): 3570 *******

DESIGN OF POROUS PAVEMENT FOR FIRST FLUSH

Project: 10.4 DESIGN OF POROUS PAVEMENT

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.58

Max. Allowable Storage Time of Aggregate Subbase(Hrs): 72

Void Ratio Aggregate Subbase: 0.4

Depth to Seasonal High Groundwater Table(Ft): 8.0

Min. Dist. from Aggr. Subbase to Groundwater Table(Ft): 2.0

***** Maximum Subbase Depth(Ft): 5.5 *****

----->>> Design Input Parameters <<<-----

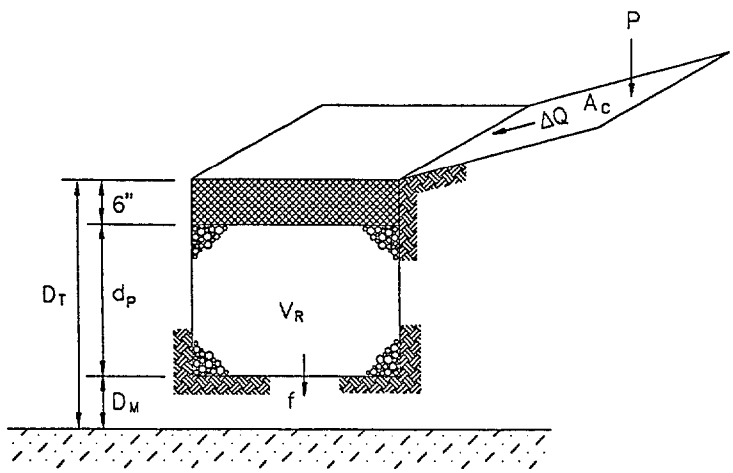
Increase in Runoff Depth(In): 0.50

Contributing Drainage Area(Sq Ft): 43560

Depth of Aggregate Subbase(Ft): 2.0

***** Porous Pavement Area(Sq Ft): 2269 *****

FIGURE 20



f = Infiltration Rate(In/Hr)

T_S = Max. Allowable Storage Time of Aggregate Subbase(Hrs)

V_R = Void Ratio Aggregate Subbase

D_T = Depth to Seasonal High Groundwater Table(Ft)

D_M = Min Dist from Aggr. Subbase to Groundwater Table(Ft)

A_C = Contributing Drainage Area(Sq Ft)

ΔQ = Increase in Runoff Depth(In)

d_P = Depth of Aggregate Subbase(Ft)

P = Amount of Rainfall(In)

T = Aggr. Subbase Reservoir Filling Time(Hrs)

For First Flush Design:

$$\text{Area of Pavement} = \frac{\frac{\Delta Q}{12} A_c}{V_R\, d_P}$$

$$\text{Area of Pavement} = \frac{\frac{\Delta Q}{12} A_c}{V_R\, d_P - \frac{P}{12} + \frac{f}{12} T}$$

POROUS PAVEMENT SCHEMATIC

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10.5 Maintenance Requirements

The surface of porous asphalt pavement must be cleaned regularly to avoid it becoming clogged by fine material. This cleaning is best accomplished through use of a vacuum cleaning street sweeper. Outside of regular cleaning, porous pavement requires no more maintenance than conventional pavement. In times of heavy snowfall it must be recognized that application of abrasive material should be closely monitored to avoid clogging problems once the snow and ice has melted. No method of maintenance has been satisfactory on fully clogged pavements, and only a superficially clogged section showing a water penetration of 0.38 inches per second can be restored to normal operation. The best method for cleaning is brush and vacuum sweeping followed by high pressure water washing of the pavement. Vacuum cleaning alone, once the pavement is clogged, has been found to be ineffective. The oils in the asphalt bind dirt, and only an abrading and washing technique can be effective in the removal of such dirt. Clogging to a depth of 0.5 inch is sufficient to prevent water penetration.

10.6 Life Expectancy

Very limited data is presently available to assess the useful life expectancy of porous pavement as a BMP. Major elements in the longevity of porous pavements are proper construction, inspection, and routine maintenance. The porous pavement surface must be vacuum swept routinely to keep the asphalt pores open. Life expectancy of six (6) months to two (2) years of proper functioning of porous pavement as a BMP is a good estimate.

10.7 Cost

Porous pavement costs should be considered as incremental costs or extra costs, incurred over and above the cost of installing a conventional parking lot. Extra costs are associated with the additional depth of the reservoir base course, porous asphalt, filter fabric and sediment and erosion control. A typical porous pavement of 3000 sq. ft. of surface area with 1.5 feet of aggregate subbase could cost an extra $5,000-6,000 over a conventional pavement. Some savings may be realized in the reduced cost of the conventional storm drainage system.

10.8 Construction Methods and Specifications

(Adapted from the Construction Specifications of the City of Rockville, MD.)

10.8.1 Stabilization

To preclude premature clogging and/or failure of this practice, porous asphalt paving structures should not be placed into service until all of the surface drainage areas contributing to the pavement have been effectively stabilized in accordance with Virginia Standards and Specifications for Soil Erosion and Sediment Control.

10.8.2 Subgrade Preparation

Alter and refine the grades as necessary to bring subgrade to required grades and sections as shown in the drawings.

The type of equipment used in subgrade preparation construction shall not cause undue subgrade compaction. (Use tracked equipment or oversized rubber tire equipment - DO NOT use standard rubber tired equipment.) Traffic over subgrade should be kept at a minimum. Where fill is required, it shall be compacted to a density equal to undisturbed subgrade, and inherent soft spots corrected.

10.8.3 Aggregate Base Course

All stone used shall be clean, washed, crushed stone, meeting Virginia Department of Transportation (VDOT) specifications.

Aggregate shall be of two sizes: the stone reservoir base course shall be to the depths noted on the drawings (maximum three-inch (3"), minimum 1.5 inch, and a two-inch (2") deep top course of 1/2" aggregate).

The stone reservoir base course shall be laid over the bottom filter course to the depths shown in the drawings, in lifts to lay naturally compacted. The stone reservoir base course shall be compacted lightly. Keep the stone reservoir base course clean from debris and sediment.

10.8.4 Porous Asphalt Surface Course

The surface course shall be laid directly over the 1/2" aggregate base course and shall be laid in one lift.

The laying temperature shall be between 230° and 260°, with minimum air temperature of 50°F, to assure that the surface does not cool prior to compaction.

Compaction of the surface course shall be done while the surface is cool enough to resist a 10-ton roller. One (1) or two (2) passes by the roller is all that is required for proper compaction. More rolling could cause a reduction in the surface course porosity.

The mixing plant shall certify the aggregate mix and abrasion loss factor and the asphalt content in the mix. The asphaltic mix shall be tested for its resistance to stripping by water use according to ASTM D 1664. If the estimated coating area is not above 95 percent, anti-stripping agents shall be added to the asphalt.

Transporting of the mix to the site shall be in clean vehicles with smooth dump beds that have been sprayed with a non-petroleum release agent. The mix shall be covered during transportation to control cooling. It should be pointed out that the mix is required to be covered during transportation by Virginia Law.

The mix of asphalt shall be 5.5 to six (6) percent of weight of dry aggregate.

Asphalt grade shall meet AASHTO Specification M-20 for 85 to 100 penetration road asphalt as a binder in the northern United States, 65 to 80 in the middle states (Virginia), and 50 to 65 in the South.

Aggregate grading shall be as specified in Table 15.

Table 15 - Porous (Open-Graded) Asphalt Concrete Formulation						
				Probable Particle Data		
Material	Screen	Weight, %	Volume, %	Width, MM	Weight, g	No. in 100g of Asphalt Concrete
Aggregate	Through 1/2	2.8	2.2	10.7	1.667	1.7
	Through 3/8	59.6	46.3	8.0	.697	85.5
	Through #4	17.0	15.3	4.0	.087	195.4
Sub-total Coarse Aggregate		79.4	61.8			282.6
	Through #8	2.8	2.2	2.0	.0109	255.6
	Through #16	10.4	8.0	1.0	.00136	7647.0
	Through 200	1.9	1.5	.06	.000294	6462.0
Asphalt		5.5	10.5			
Air		0	16.0			
		100.0	100.0			
Source: City of Rockville, Maryland (1982)						

10.8.5 <u>Protection</u>

After final rolling, no vehicular traffic of any kind should be permitted on the pavement until cooling and hardening has taken place, and in no case less than six (6) hours (preferably a day or two).

10.8.6 <u>Workmanship</u>

Work shall be done expertly throughout and without staining or damage to other permanent work. Make transition between existing and new paving work neat and flush. Finished paving shall be even, without pockets, and graded to elevations shown. Iron smoothly to grade, all minor surface projections and edges adjoining other materials.

11 GRID/MODULAR PAVEMENT

11.1 Description

Grid/Modular pavement as a BMP is similar to other infiltration practices. Instead of a conventional pavement, a pervious pavement consisting of a grid made of concrete, clay bricks, or granite sets can be constructed. Void areas of the grid pavement can be filled with pervious material like sand, gravel, or sod.

There are three categories of grid paving material. They are based on their surface configurations, and are listed in Table 16. Representative grid pavements are shown on Figure 21.

Table 16 - Types of Grid Pavements

Type	Configuration	Brand Names
Lattice Pavers	Flat Grid-like	Unigreen, Turfstone, Grasstone
Castellated Pavers	Raised Battlements	Monoslab, checkerblock
Poured-in-place Pavers	Flat grid-like	Grasscrete

A schematic of Grid/Modular pavement is shown on Figure 22. Grid/Modular pavement detail is shown on Figure 23.

11.2 Applicability

Grid/Modular pavements should be used for areas with light traffic and less frequently travelled parking lots. Another use can be for walkways in recreational areas. All driveways in residential areas, auxiliary parking, emergency fire lanes etc. can be good applications of grid/modular pavements.

11.3 Design Criteria

The design criteria for grid/modular pavement is similar to the infiltration trench criteria. It is included here for ease of use.

11.3.1 Soil Permeability

Soil textural classes with infiltration rates greater than or equal to 0.27 inches per hour should be used for the installation of grid/modular pavements. This infiltration rate is associated with soil textural groups of sand, loamy sand, sandy loam, loam, and silt loam. The infiltration rate of the underlying soil and the depth of the groundwater table are the major limiting factors in the selection and feasibility of the grid/modular pavement as a BMP.

11.3.2 Depth of Subbase

The final infiltration rate of the soil below the infiltration trench determines the maximum allowable subbase depth. The surface area of the grid/modular pavement can be minimized by making the subbase as deep as feasible. The subbase of the grid/modular pavement can also be made shallow and broad. The increased surface area of the bottom of the subbase increases exfiltration rates and provides more area for soil filtering of pollutants. A larger subbase bottom also helps in reducing clogging by providing exfiltration over a wide area.

11.3.3 Groundwater Table

The seasonal high groundwater table should be located at least two (2) to four (4) feet below the bottom of the subbase of the grid/modular pavement. The soil permeability (infiltration rate) and the groundwater table are the two parameters which determine the maximum allowable depth of the grid/modular pavement.

11.3.4 Proximity To Wells and Foundations

Grid/Modular pavements should be located at least 100 feet upgradient from any drinking water supply well to minimize the possibility of groundwater contamination. Also the grid/modular pavements should be located at least ten (10) feet downgradient and 100 feet upgradient from building foundations.

11.3.5 Design Storm

Grid/Modular pavements can be designed for a specific storm or for the first flush runoff volume. If for first flush, the subbase storage volume can be sized based on 0.5 inches of runoff per impervious acre in the contributing site area.

Grid Modular pavements are normally designed for water quality. As such, a significant portion of the runoff volume (storms producing more than 0.5 inches of runoff) will bypass the grid/modular pavement and is not infiltrated.

11.3.6 Storage Time/Maximum Draining Time

All grid/modular pavements should be designed to drain within a maximum time of three (3) days (72 hours), or a minimum time of two (2) days (48 hours). These values are derived from literature. The Virginia Stormwater Management Regulations recommend two (2) days (48 hours).

11.3.7 Stone Aggregate

The stone aggregate which fills the grid/modular pavement forms the reservoir through which the storm runoff passes and is filtered. The aggregate material should be clean, washed stone. Wash run gravel is preferred. The City of Virginia Beach recommends using James River stone as aggregate. The clean washed stone aggregate should have a maximum diameter of three (3) inches and a minimum diameter of one (1) inch. Void spaces for the stone aggregate are normally within the range of 30 to 40 percent. A table showing open graded coarse aggregates is included in the Appendix.

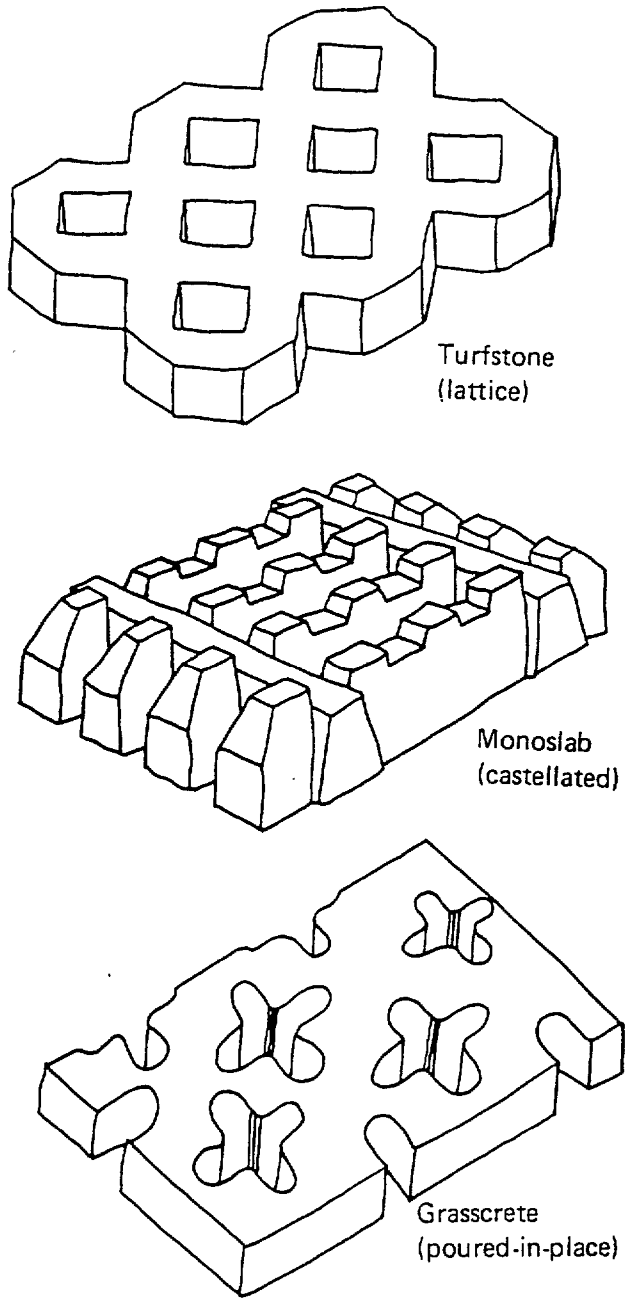
11.3.8 Runoff Filtering

It is important to prevent any floatable material, settleable solids, grease, and oil from entering the grid/modular pavement. Runoff filtering devices such as vegetative filter strips (minimum of 20 feet) can be used in front of the grid/modular pavement to prevent objectionable materials from entering the subbase.

11.3.9 Overflow Requirements

In all cases, the overflow path of storm runoff exceeding the capacity of the subbase of the grid/modular pavement should be evaluated and accommodated. The grid/modular pavements are designed to treat the first flush volume of runoff and control small drainage areas.

FIGURE 21



REPRESENTATIVE GRID PAVEMENTS

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11.4 Design Example

Design modular pavement for the driveway of a single-family residential lot. The driveway size is 24 feet x 40 feet. Associated runoff for a 1.5-inch storm is 0.6 inch. Underlying soil is border-line sandy loam with an infiltration rate of 0.85 inches per hour. Depth to groundwater is three (3) feet. Pavement subbase depth should be one (1) foot.

DESIGN OF GRID / MODULAR PAVEMENT

Project: 11.4 DESIGN EXAMPLE FOR A DRIVEWAY

----->>> Feasibility Input Parameters <<<-----

Infiltration Rate(In/Hr): 0.85

Maximum Allowable Storage Time of Pavement(Hrs): 48

Void Ratio Subbase Medium: 0.4

Depth to Seasonal High Groundwater Table(Ft): 3.0

Min. Dist. from Subbase bottom to Groundwater Table(Ft): 2.0

***** Maximum Subbase Depth(Ft): 1.0 *****

----->>> Design Input Parameters <<<-----

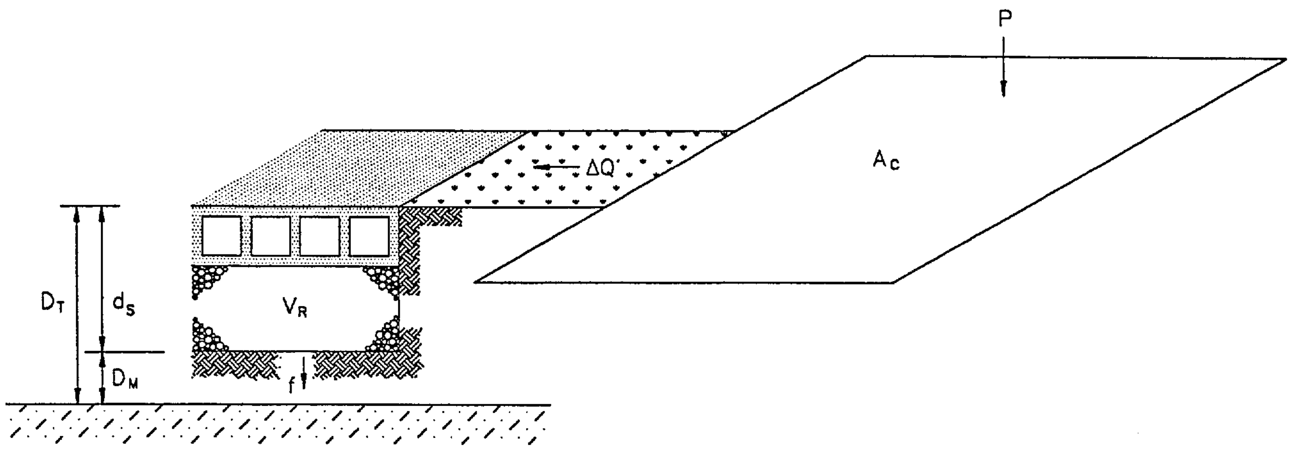
Increase in Runoff Depth(In): 0.60

Contributing Drainage Area(Sq Ft): 960

Depth of Subbase(Ft): 1.0

***** Pavement Area(Sq Ft): 120 *****

FIGURE 22



f = Infiltration Rate(In/Hr)

T_S = Maximum Allowable Storage Time of Pavement(Hrs)

V_R = Void Ratio Subbase Medium

D_T = Depth to Seasonal High Groundwater Table(Ft)

D_M = Min Dist from Subbase bottom to Groundwater Table(Ft)

A_C = Contributing Drainage Area(Sq Ft)

ΔQ = Increase in Runoff Depth(In)

d_S = Depth of Subbase(Ft)

$$\text{Area of Pavement} = \frac{\frac{\Delta Q}{12} A_C}{V_R\, d_S}$$

GRID/MODULAR PAVEMENT SCHEMATIC

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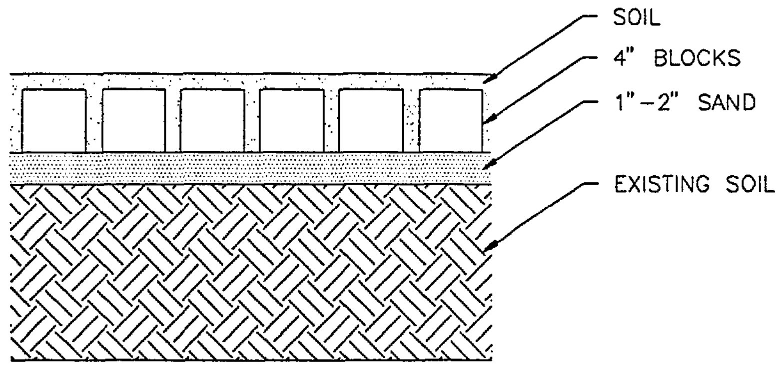
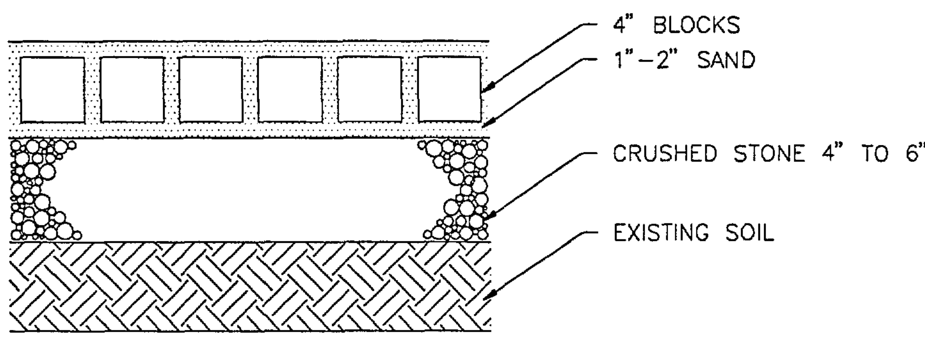
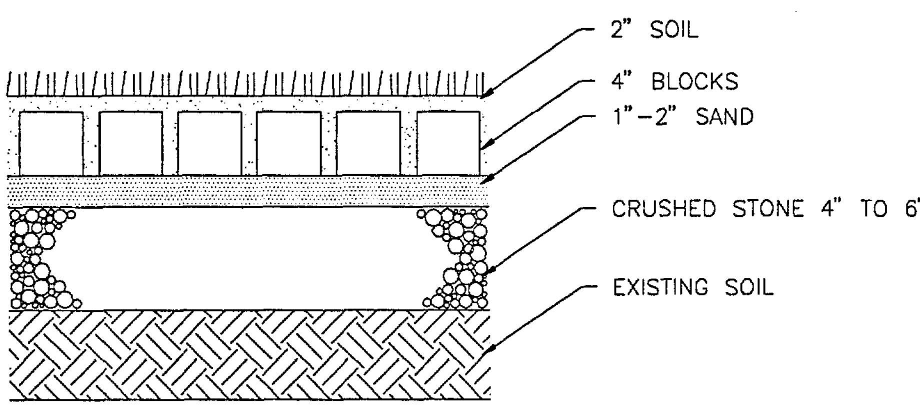
FIGURE 23



AUTO DRIVEWAY



TRUCK DRIVEWAY



ROADS UNDER LAWN

MODULAR PAVEMENT DETAIL

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11.5 Maintenance Requirements

Grid/Modular pavements, if constructed properly, should have minimal maintenance requirements. Routine maintenance will involve regular mowing of the grass and replacing or adjusting the unequal settlement of the grid pavement. If grass is used in the void areas, the fertilizer to be used should not cause deterioration of the paving material.

11.6 Life Expectancy

Grid/Modular pavement may have a longevity from six (6) months to two (2) years. Proper construction, inspection, and maintenance are the key elements enhancing life expectancy.

11.7 Cost

The cost of installing a Grid/Modular pavement may range from three (3) to seven (7) dollars per square foot. This cost will be in addition to the installation of an infiltration trench if needed. No filter fabric is required for the top and side portion of the aggregate reservoir subbase of this BMP.

11.8 Construction Specifications

Manufacturers of brand name grid pavers have standard specifications for their product. Those specifications should be followed strictly for construction of the pavement. The specifications outlined under infiltration trench also apply for this BMP. They are included here for ease of use.

11.8.1 <u>Timing</u>

Grid/Modular pavement should not be constructed or placed in service until all of the contributing drainage area has been stabilized and approved by the responsible inspector.

11.8.2 Subbase Preparation

Excavate the subbase to the design dimensions. Excavated materials should be placed away from the excavated sides of the subbase to enhance wall stability. Large tree roots must be trimmed flush with the subbase sides. The side walls of the subbase should be roughened where sheared and scaled by heavy equipment.

11.8.3 Stone Aggregate Placement and Compaction

The stone aggregate should be placed in lifts and compacted using plate compactors. As a rule of thumb, a maximum loose lift thickness of 12 inches is recommended. The compaction process ensures conformity to the excavation sides, thereby reducing the potential for soil piping and settlement problems.

11.8.4 Contamination

Care shall be exercised to prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate must be removed and replaced with uncontaminated stone aggregate.

11.8.5 Unstable Excavation Sides

Vertically excavated walls may be difficult to maintain in areas where the soil moisture is high or where soft cohesive or cohesionless soils predominate. These conditions may require laying back of the side slopes to maintain stability; trapezoidal rather than rectangular cross sections may result.

11.8.6 Vegetative Buffer

A vegetative buffer of at least 20 feet wide (wider if possible) should be used to intercept surface runoff from all impervious areas.

11.8.7 Traffic Control

Heavy equipment and traffic shall be restricted from travelling over the infiltration areas to minimize compaction of the soil.

12 GRIT-OIL SEPARATOR

12.1 Description

Grit-oil separators are considered practical to remove hydrocarbons, coarse sediments, and grit before they are conveyed to a storm drain system. Pollutants tied to coarse sediments may also be removed. A typical grit-oil separator consists of three chambers. The first chamber receives the storm runoff through a storm drain or the opening of a curb inlet. Any grit or sediment is trapped in this chamber. Floating material like bottles, leaves, and other containers are also trapped in the first chamber. The first chamber is connected to the second chamber through a minimum of two orifices which are screened by trash racks to prevent clogging of the orifices. The orifices are located at a minimum of 18 inches above the floor of the structure. The second and third chambers are connected by an inverted pipe, and the runoff passes upward from the bottom opening of the pipe. The first two chambers maintain a permanent pool of water. The third chamber connects to the storm drain system or other infiltration BMP.

A Grit-oil separator schematic is shown on Figure 24. Plan view of a Grit-oil separator is shown on Figure 25 and details are shown on Figure 26 and Figure 27.

12.2 Applicability

Grit-oil separators are typically installed in parking lots or commercial sites of one (1) acre or less. This BMP is suitable for sites where there is vehicular traffic and a good chance of oil and grease being washed off by the storm runoff. Grit-oil separators are most suitable for sites such as convenience stores, gas stations, etc.

12.3 Design Criteria

Grit-oil separators are sized to provide a permanent pool of water or wet storage of at least 4.5 feet in depth. This depth consists of a minimum of 1.5 feet from the bottom of the structure to settle the grit and sediments and a minimum of three (3) feet above the pool to provide wet storage for oil, sediments, and objectionable floating material. The basin is sized to provide 200 cubic feet of pollutant storage per contributing acre of the site.

The length of the first chamber should be a minimum of six (6) feet. The second and third chambers should be at least four (4) feet long. The width of the structure should be a minimum of 2.5 feet. Longer lengths and greater widths can be used to reduce the height of the structure.

The total height of the structure will be determined by the head required to pass the developed condition ten-year discharge through the inverted drawdown pipe connecting the second and third chambers, if the grit-oil separator is connected to the storm drain system (on-line). For the curb inlet opening configuration, the head required to pass the storm runoff through the inverted drawdown pipe can be for the first flush discharge only.

A minimum of 1.5 feet of freeboard should be provided as a part of the total height of the structure.

The inverted drawdown pipes should be at least six (6) inches in diameter and there shall be at least two in number. Drawdown pipes larger than six (6) inches in diameter and more than two (2) in number can be used to connect the second and third chamber to reduce the total height of the grit-oil separator.

Access to each chamber for regular clean-outs and inspection should be provided through separate manhole covers. Each chamber should have steps for easy accessibility.

The walls and bottom floor of the structure should be constructed of reinforced concrete and should be structurally sound. If constructed on wet soil or where the groundwater table is high, the structures should be checked for floatation.

12.4 Design Examples

A commercial site, 13,000 square feet in size is entirely paved. Design a grit-oil separator with three (3) chambers to control the runoff from the site. The storm runoff being transported to the BMP is one (1) cubic feet per second (CFS).

DESIGN OF GRIT-OIL SEPARATOR

Project: 12.4 DESIGN FOR 0.3 ACRE COMMERCIAL SITE

----->>> Input Parameters <<<-----

Contributing Impervious Drainage Area(Ac): 0.30

Length of First Chamber(Ft): 6.00

Width of First Chamber(Ft): 2.50

Free-Board(Ft): 1.50

Contributing Flow From Impervious Area(cfs): 1.0

Diameter of Drawdown Pipe(In): 6.0

Number of Drawdown Pipes: 2

----->>> Output Values <<<-----

Oil Storage Volume(Cu Ft): 60

Depth of Oil Storage (Ft): 4.00

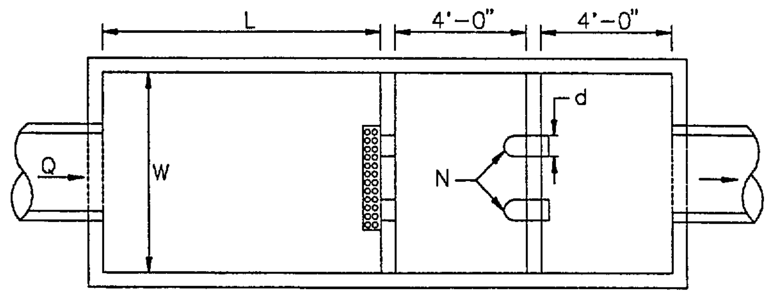
Height to Pass Flow through Drawdown Pipe(Ft): 0.28

First Chamber(L x W x H): (6.00' x 2.50' x 7.28')

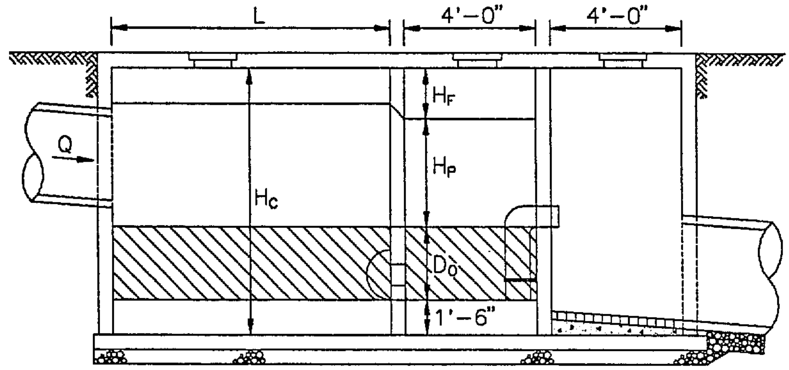
Second Chamber(L x W x H): (4.00' x 2.50' x 7.28')

Third Chamber(L x W x H): (4.00' x 2.50' x 7.28')

FIGURE 24



TOP VIEW



SIDE VIEW

A_C = Contributing Drainage Area(Acres)

L = Length of Chamber(Ft)

W = Width of Chamber(Ft)

H_F = Free–Board(Ft)

Q = Contributing Flow From Impervious Area(cfs)

d = Diameter of Drawdown Pipe(In)

N = # of Drawdown Pipes

H_P = Height to Pass Flow through Drawdown Pipe(Ft)

H_C = Overall Height of Chamber(Ft)

D_O = Depth of Oil Storage(Ft)

V = Volume of Oil Storage(Cu Ft)

$V = 200\ A_C$ $\quad D_O = \frac{V}{L\ W}$ $\quad H_P = \frac{0.07\ Q^2}{\left(\frac{d}{12}\right)^4 N^2}$ $\quad H_C = 1.5 + D_O + H_P + H_F$

GRIT-OIL SEPARATOR SCHEMATIC

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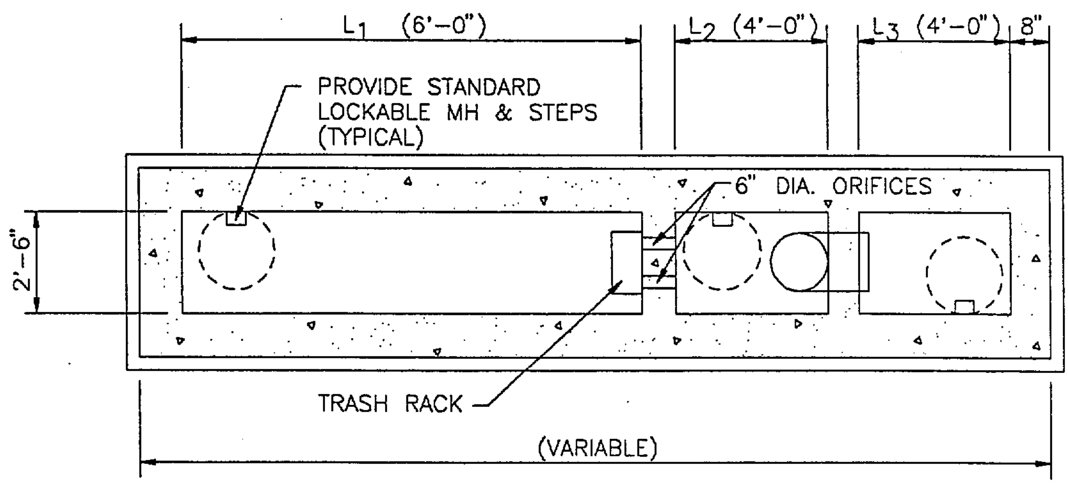
FIGURE 25



PLAN VIEW

GRIT-OIL SEPARATOR DETAIL

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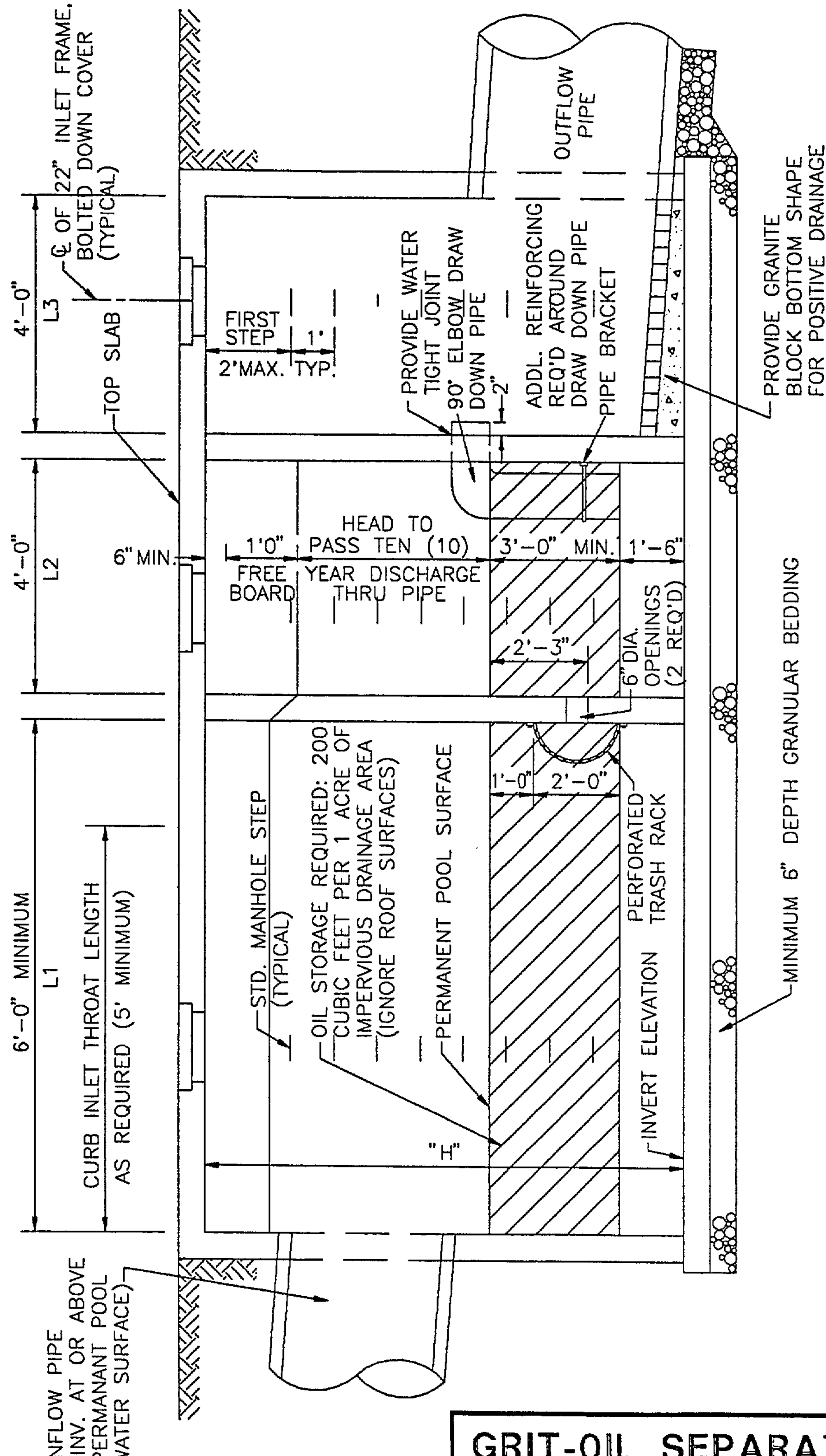
FIGURE 26



GRIT-OIL SEPARATOR DETAIL

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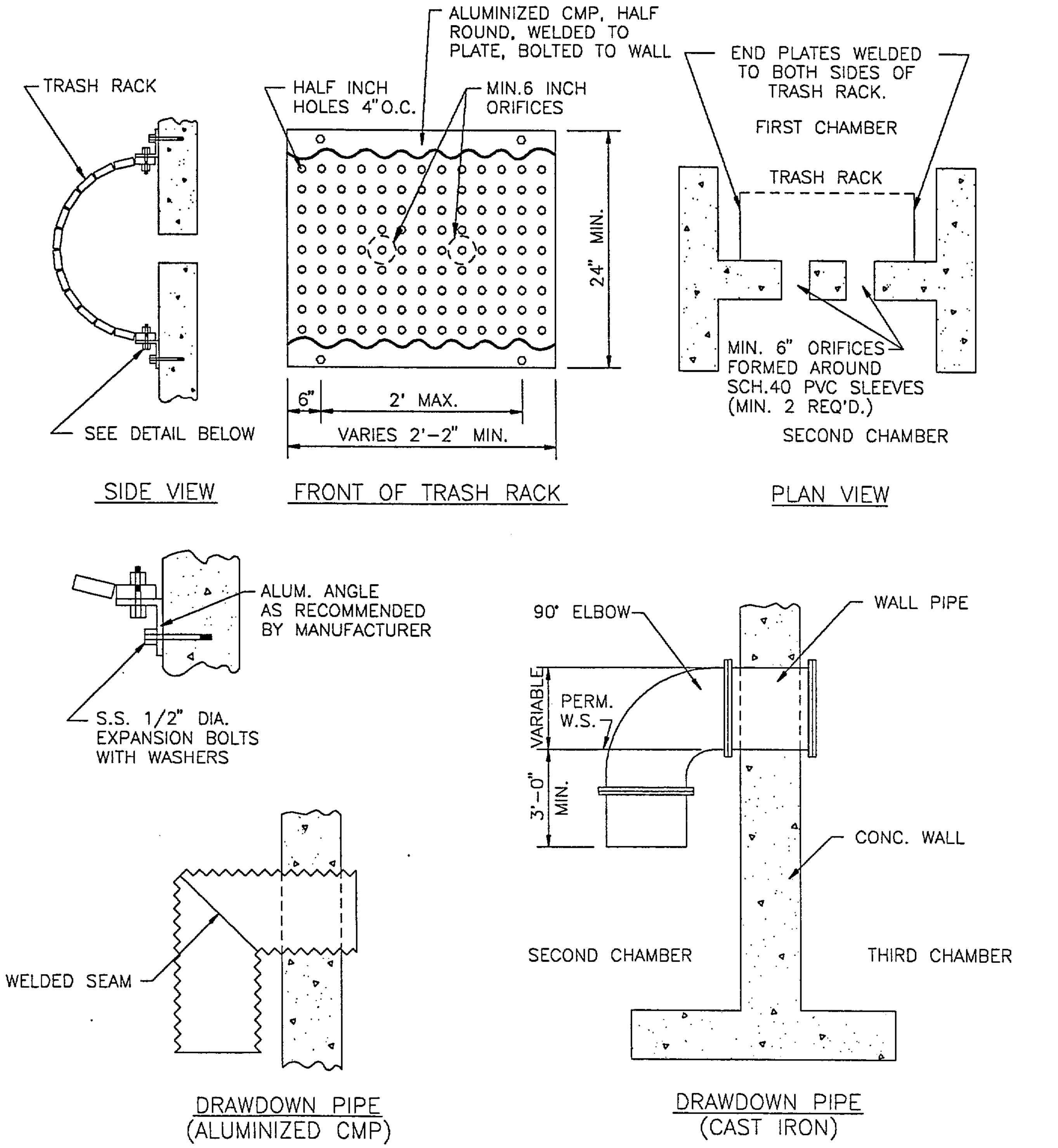
FIGURE 27



GRIT-OIL SEPARATOR DETAIL

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12.5 Maintenance Requirements

Grit-oil separators function properly only if they are cleaned regularly, at least twice a year. Oil soaked grit and sediments, and oily sheen slurry in the first and second chamber should be pumped out and properly disposed. Material should be dried and tested for toxicity according to Southeastern Public Service Authority (SPSA) of Virginia approved procedures. The dry material, if non-toxic, can be disposed of at the regional landfill with prior approval from SPSA.

12.6 Life Expectancy

Grit-oil separators have not been in use for a long enough time to accurately estimate their life expectancy. If routinely cleaned and maintained, they can function properly like any storm drain manhole and may last 20 years. More monitoring data needs to be gathered before making an accurate estimate of their longevity.

12.7 Cost

There are significant costs associated with this BMP. They can range from \$5,000 - \$20,000 for each installation depending on the size of the structure. Pre-cast versions of the concrete chambers may lower the total construction cost.

12.8 Construction Specifications

The materials used for installing a grit-oil separator are associated with normal storm drain construction. Typical storm drainage specifications can be easily utilized for this BMP. It is recommended to refer to local jurisdictions' storm drain design manuals for these specifications.

13 WATER QUALITY INLET

13.1 Description

The water quality inlet is a smaller version of a grit-oil separator and functions in a similar fashion. Instead of a three chamber design, a water quality inlet can consist of only one or two chambers. The first and second chambers are connected through the inverted drawdown pipe. The outflow from the second chamber flows to the storm drain system or other infiltration BMP.

A water quality inlet schematic is shown on Figure 28. Details for the water quality inlet are shown on Figure 29 and Figure 30.

13.2 Applicability

Water quality inlets should be used for small commercial sites or as pretreatment facilities for other infiltration BMPs. The outflow from the inverted drawdown pipe can be directly transported to an infiltration facility, thus eliminating the second chamber.

13.3 Design Criteria

Water quality inlets are sized to provide a permanent pool of water or wet storage of at least 4.5 feet in depth. This depth consists of a minimum of 1.5 feet from the bottom of the structure to settle the grit and sediments, and a minimum of three (3) feet above the pool to provide wet storage for oil, sediments, and objectionable floating material. The basin is sized to provide 200 cubic feet of pollutant storage per contributing acre of the site.

The design criteria listed under grit-oil separator are also applicable to the water quality inlet. Water Quality Inlets are sized to provide a permanent pool of water or wet storage of at least 4.5 feet in depth. This depth consists of a minimum of 1.5 feet from the bottom of the structure to settle the grit and sediments and a minimum of three (3) feet above the pool to provide wet storage for oil, sediments, and objectionable floating material. The basin is sized to provide 200 cubic feet of pollutant storage per contributing acre of the site.

Instead of three chambers, a water quality inlet may have one or two chambers. The length of the first chamber should be a minimum of six (6) feet. The second chamber should be at least four (4) feet long. The width of the structure should be a minimum of 2.5 feet. Longer lengths and greater widths can be used to reduce the height of the structure.

The total height of the structure will be determined by the head required to pass the developed condition ten-year discharge through the inverted drawdown pipe connecting the first and second chambers, if the water quality inlet is connected to the storm drain system (on-line). For the curb inlet opening configuration, the head required to pass the storm runoff through the inverted drawdown pipe can be for the first flush discharge only.

A minimum of 1.5 feet of freeboard should be provided as a part of the total height of the structure.

The inverted drawdown pipes should be at least six (6) inches in diameter and there should be at least two in number. Drawdown pipes larger than six (6) inches and more than two (2) in number can be used to connect the first and second chamber to reduce the total height of the water quality inlet.

Access to each chamber for regular clean-outs and inspection should be provided through separate manhole covers. Each chamber should have steps for easy accessibility.

The walls and bottom floor of the structure should be constructed of reinforced concrete and should be structurally sound. If constructed on wet soil or where the groundwater table is high, the structures should be checked for floatation.

13.4 Design Example

A commercial site, 13,000 square feet in size is all paved. Design a water quality inlet with two (2) chambers to control the runoff from the site. The storm runoff being transported to the BMP is one (1) cubic feet per second (CFS).

DESIGN OF WATER QUALITY INLET

Project: 12.5 DESIGN FOR 13000 SQ.FT. COMMERCIAL SITE

----->>> Input Parameters <<<-----

Contributing Impervious Drainage Area(AC): 0.300

Length Chamber(Ft): 6.00

Width Chamber(Ft): 2.50

Free-Board(Ft): 1.50

Contributing Flow From Impervious Area(cfs): 1.0

Diameter of Drawdown Pipe(In): 6.0

Number of Drawdown Pipes: 2

----->>> Output Values <<<-----

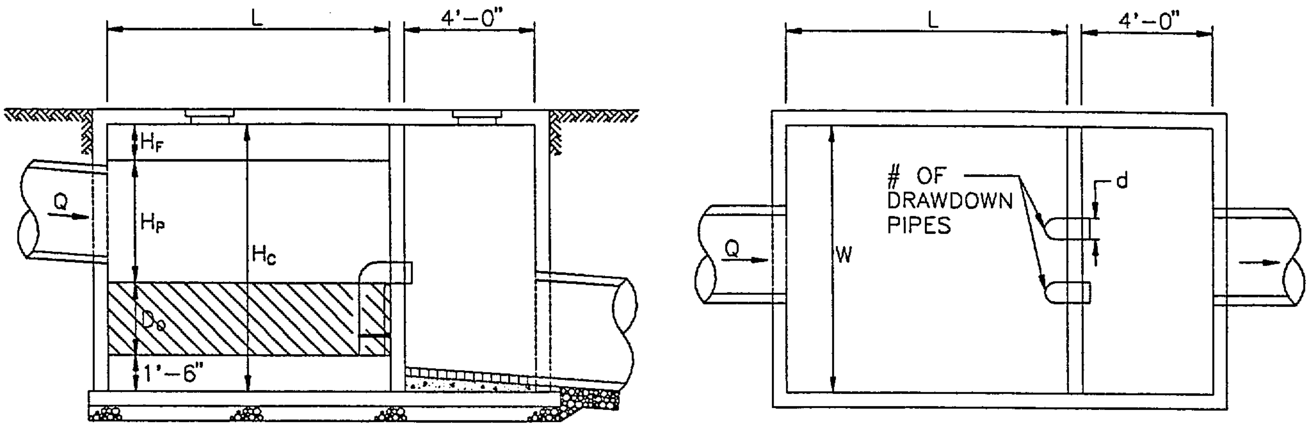
Oil Storage Volume(Cu Ft): 60

Depth of Oil Storage (Ft): 4.00

Height to Pass Flow through Drawdown Pipe(Ft): 0.28

First Chamber(L x W x H): (6.00' x 2.50' x 7.28')

Second Chamber(L x W x H): (4.00' x 2.50' x 7.28')



SIDE VIEW TOP VIEW

A_C = Contributing Drainage Area(Acres)

L = Length of Chamber(Ft)

W = Width of Chamber(Ft)

H_F = Free–Board(Ft)

Q = Contributing Flow From Impervious Area(cfs)

d = Diameter of Drawdown Pipe(In)

N = # of Drawdown Pipes

H_P = Height to Pass Flow through Drawdown Pipe(Ft)

H_C = Overall Height of Chamber(Ft)

D_O = Depth of Oil Storage(Ft)

V = Volume of Oil Storage(Cu Ft)

$$V = 200\,A_C \qquad D_O = \frac{V}{L\,W} \qquad H_P = \frac{0.07\,Q^2}{\left(\frac{d}{12}\right)^4 N^2} \qquad H_C = 1.5 + D_O + H_P + H_F$$

WATER QUALITY INLET SCHEMATIC

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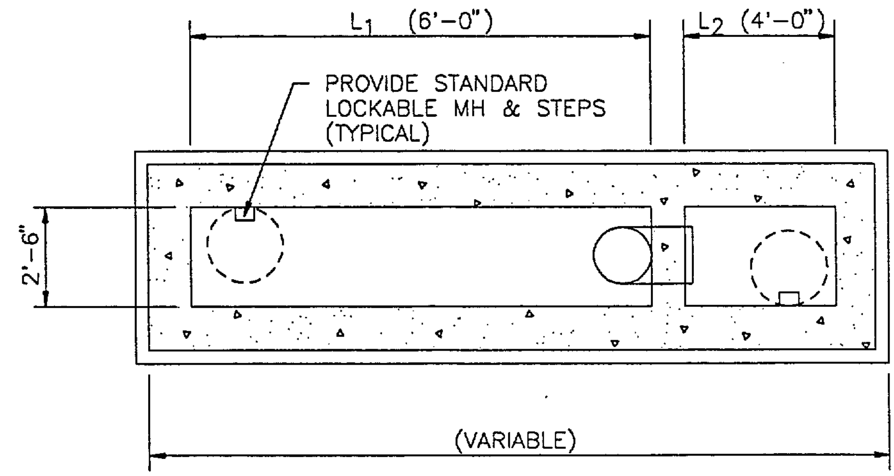
FIGURE 29



PLAN VIEW

WATER QUALITY INLET DETAIL

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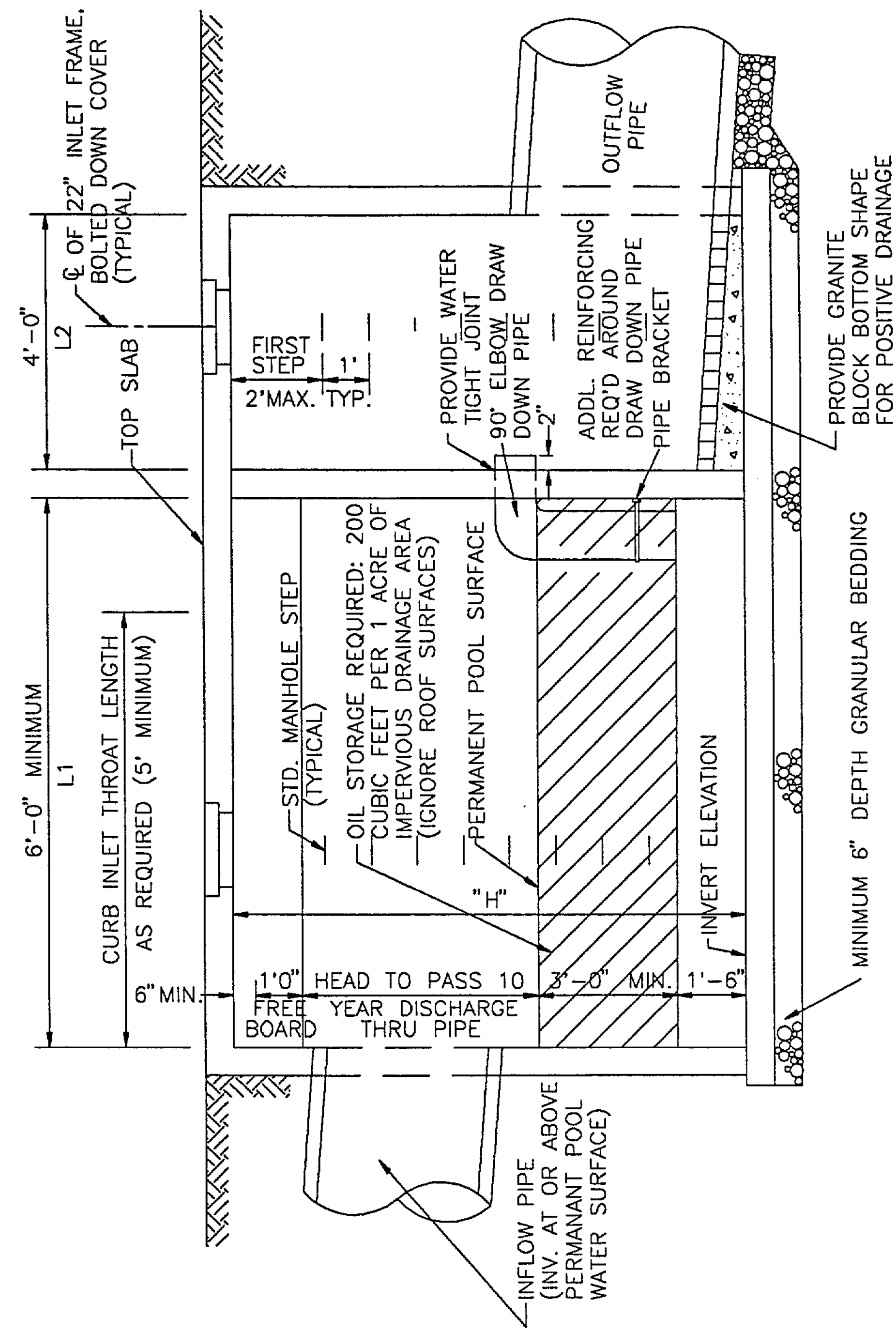
FIGURE 30



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13.5 Maintenance Requirements

Water quality inlets function properly only if they are cleaned regularly, at least twice a year. Oil soaked grit and sediments, and oily sheen slurry in the first and second chamber should be pumped out and properly disposed. Material should be dried and tested for toxicity according to Southeastern Public Service Authority (SPSA) of Virginia approved procedures. The dry material, if non-toxic, can be disposed of at the regional landfill with prior approval from SPSA.

13.6 Life Expectancy

Water quality inlets have not been in use for a long enough time to accurately estimate their life expectancy. If routinely cleaned and maintained, they can function properly like any storm drain manhole and may last 20 years. More monitoring data needs to be gathered before making an accurate estimate of their longevity.

13.7 Cost

Since a water quality inlet is smaller than a grit-oil separator, the cost will be slightly less. A typical two chamber structure may range from $3,000 to $8,000. The installation cost of another BMP if used in combination, is an additional cost.

13.8 Construction Specifications

The materials used for installing a water quality inlet are associated with normal storm drain construction. Typical drainage specifications can be easily utilized for this BMP. It is recommended to refer to local jurisdictions' storm drain design manuals for these specifications.

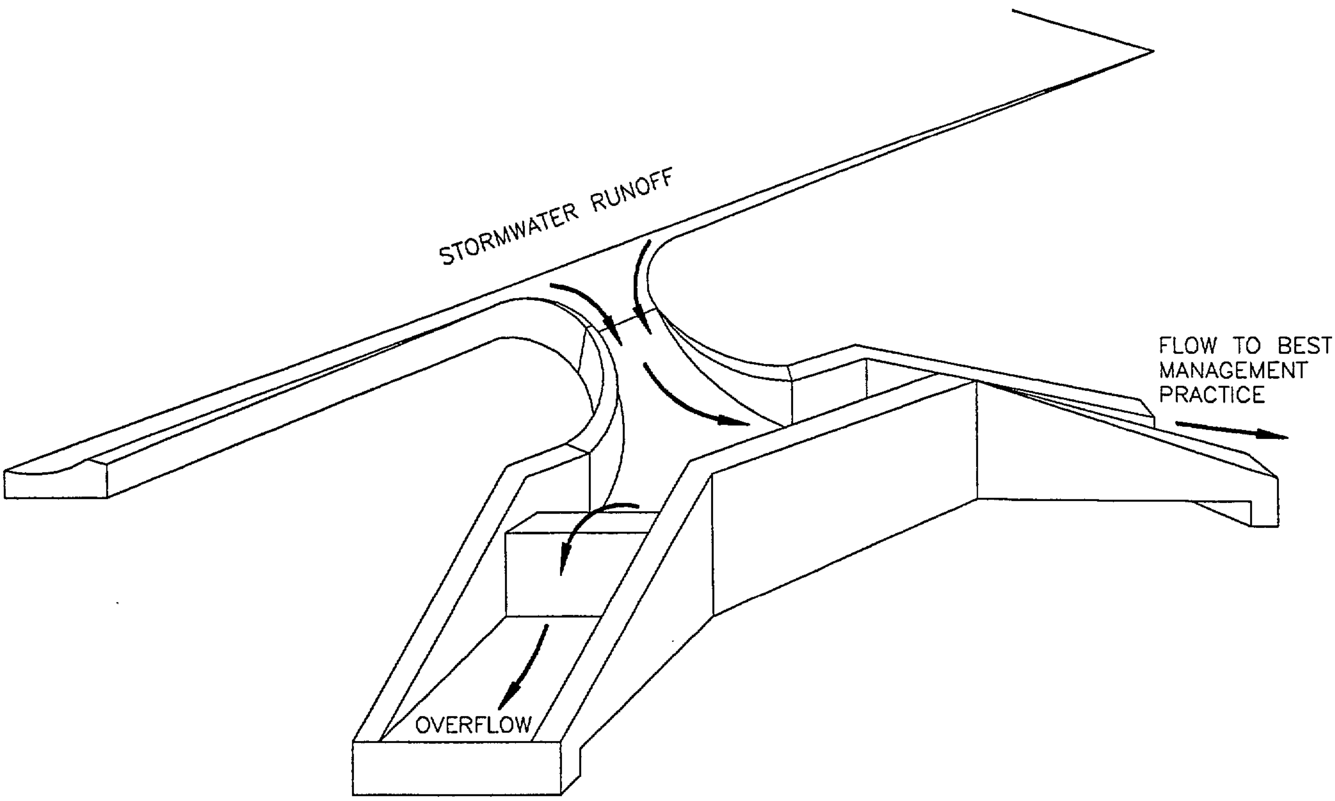
14 BMP COMBINATIONS

All BMPs are designed to receive the first flush of runoff volume during a storm. The remaining runoff volume is not treated by the BMP, and is conveyed to the storm drain system or a detention or retention pond downstream. The separation of the first flush and the remaining runoff can be accomplished by constructing a weir. Height of the weir can be designed to divert the first flush to a BMP and the overflow to a storm drain or downstream SWM facility. This concept is shown on Figures 31 and 32.

Exfiltration of the runoff volume can be increased by constructing an infiltration trench under a grassed swale. Figure 33 illustrates the concept. Porous pavement can be combined with a surface infiltration trench and grass filter strip. Runoff can be evenly distributed to the infiltration trench by placing slotted curbs at the end of the porous pavement and by constructing a berm of Austin triangle at the end of the grass filter strip. A schematic of this combination is shown on Figure 34. Figure 35 shows another possible combination of grassed swale and infiltration trench. For design criteria and other pertinent information, refer to individual chapters on each BMP.

Commercial/industrial parking lots generate significant loads of grit and oil. If infiltration practices are used to treat surface runoff, they will be rapidly clogged. Surface runoff needs to be pretreated to remove all objectionable material before it enters the infiltration practices. Figures 36 to 38 show possible combinations of grit/oil separators and water quality inlets for pretreating the runoff. Individual chapters on these BMPs describe the design criteria and other pertinent information.

FIGURE 31



CONCRETE DIVERSION DETAIL

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HAMPTON ROADS
PLANNING DISTRICT COMMISSION

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CHK'D:
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DATE:

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CHK'D: V.P.M.
DATE: OCT. 1990
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FIGURE 32

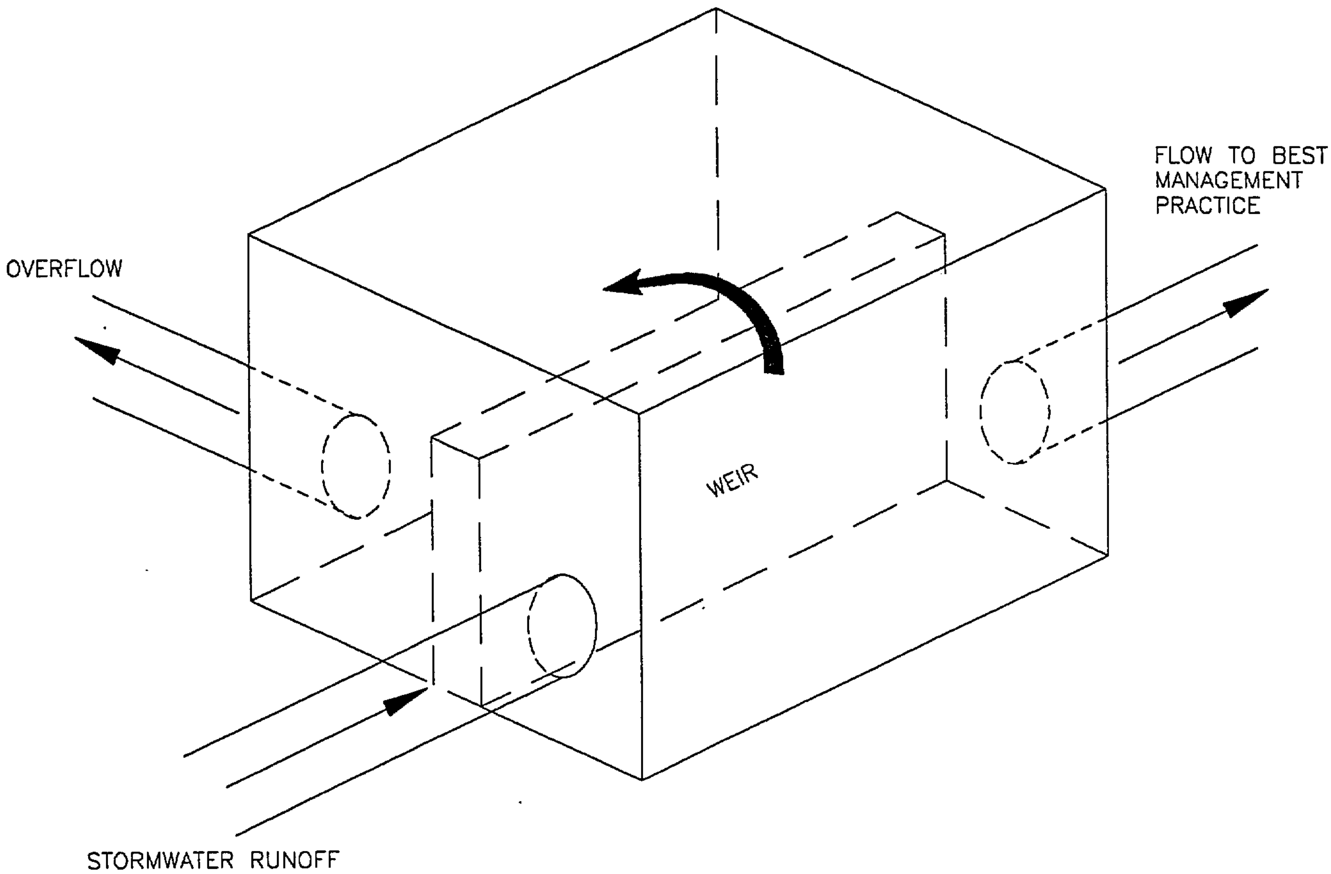

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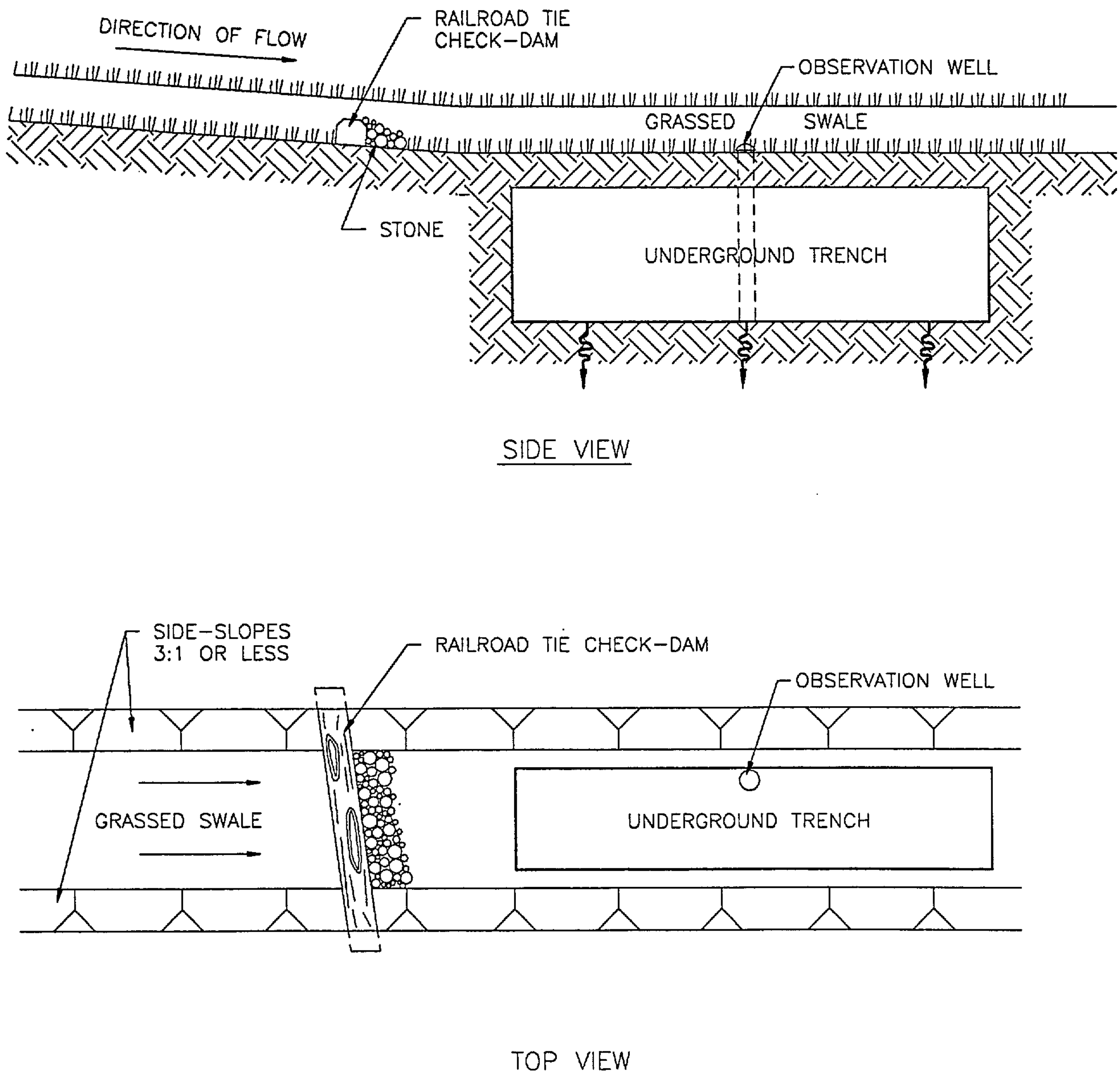
FIGURE 33



UNDER-THE-SWALE TRENCH

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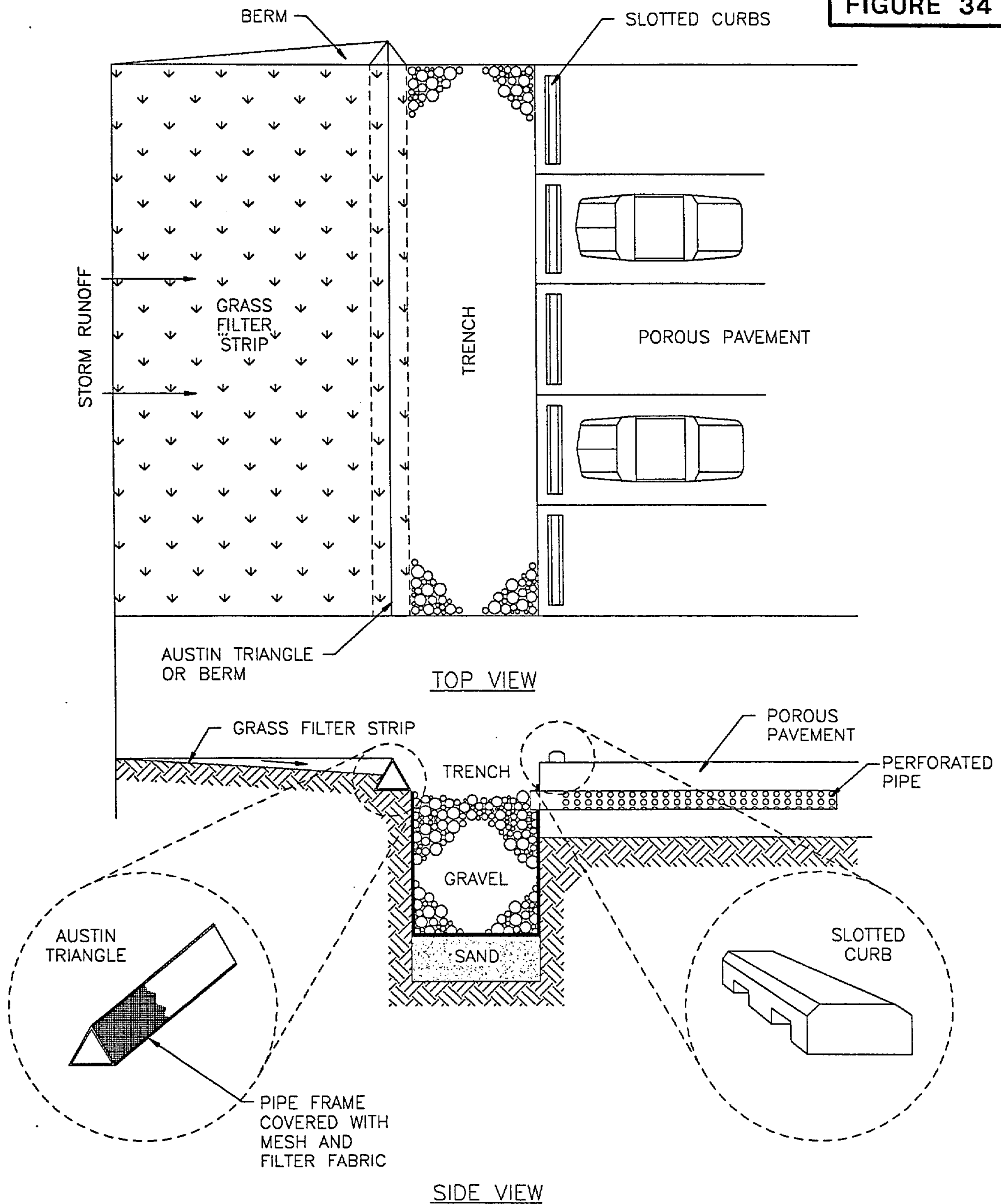
FIGURE 34



FILTER STRIP POROUS PAVEMENT

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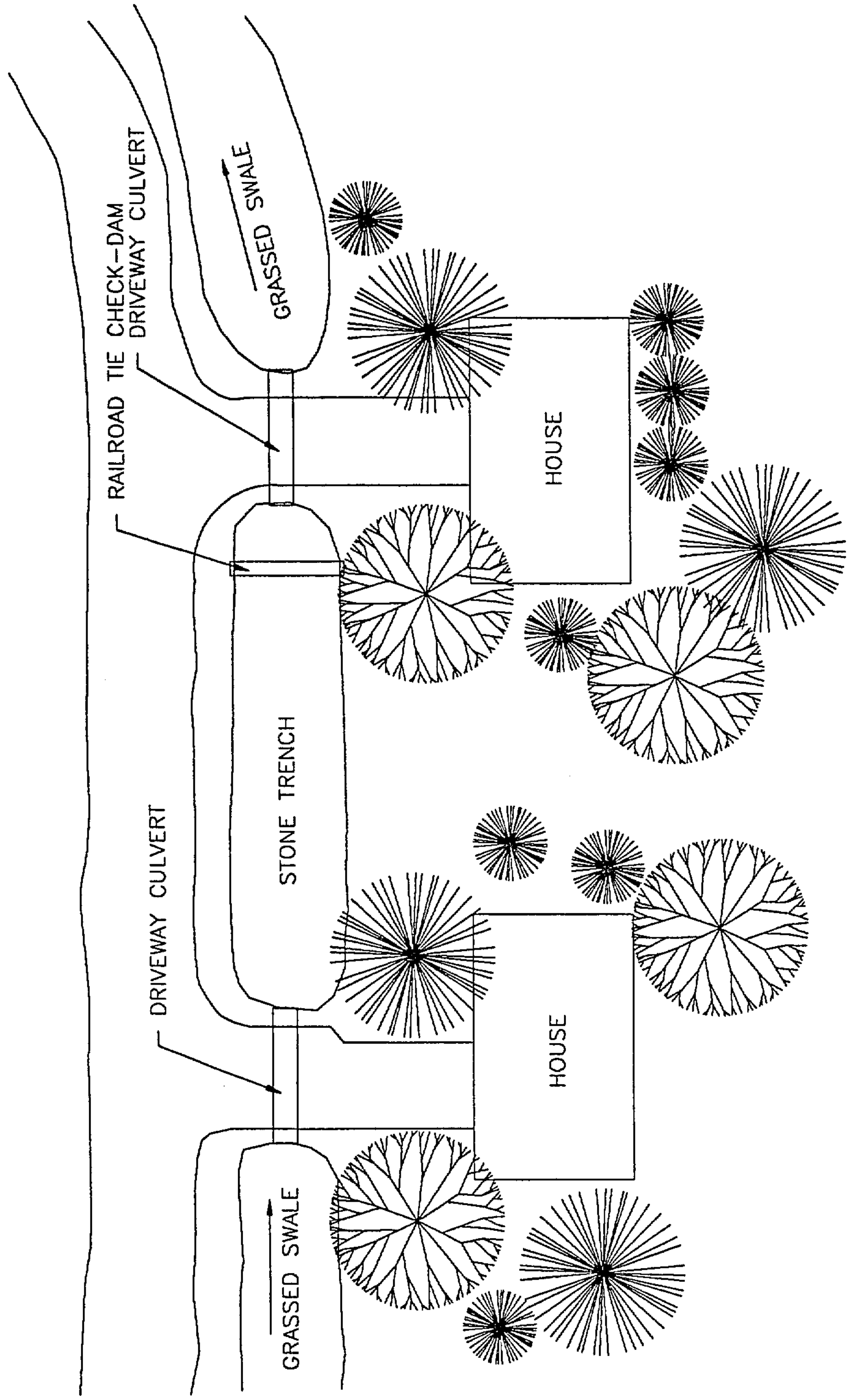
FIGURE 35



SWALE/TRENCH

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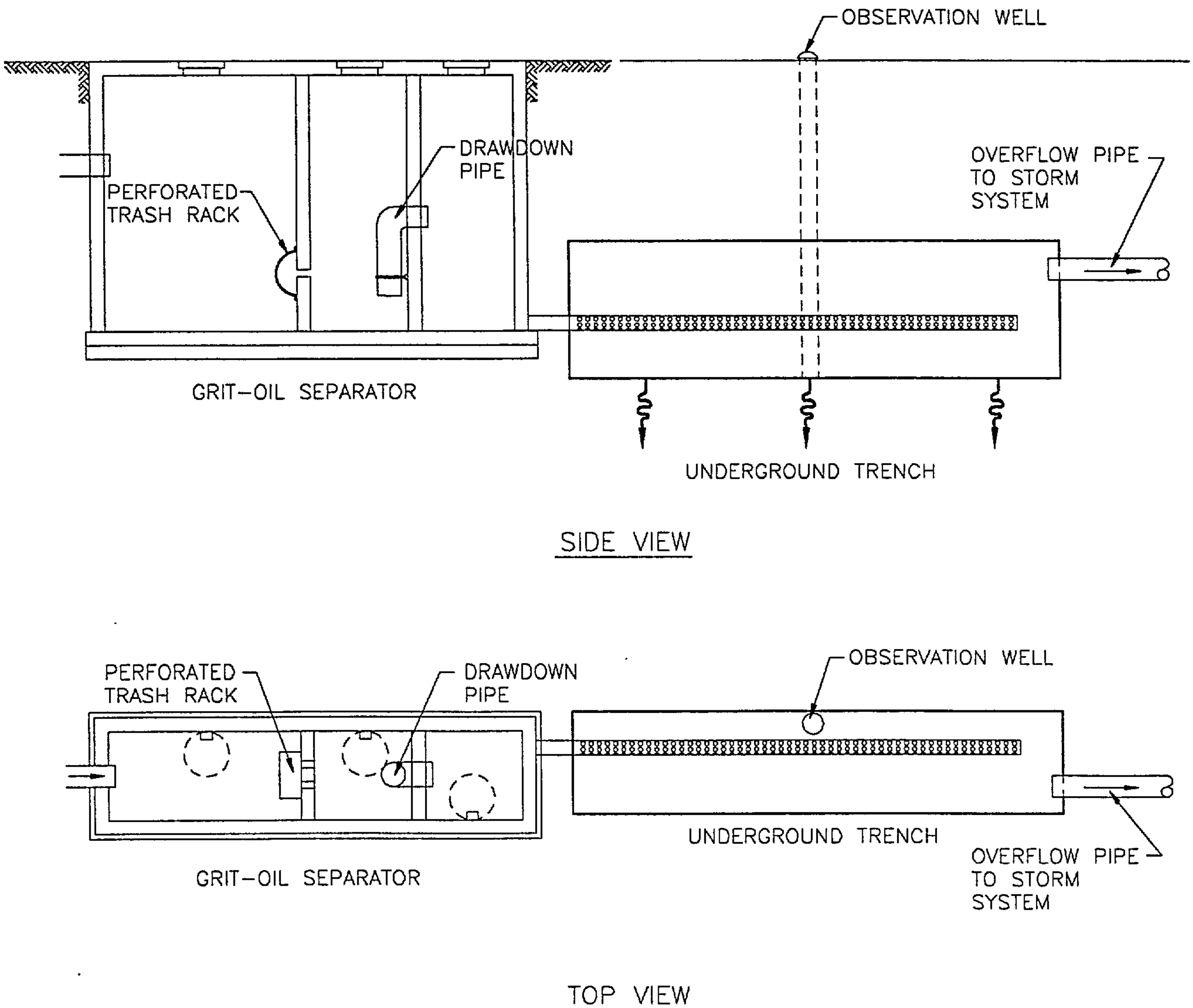
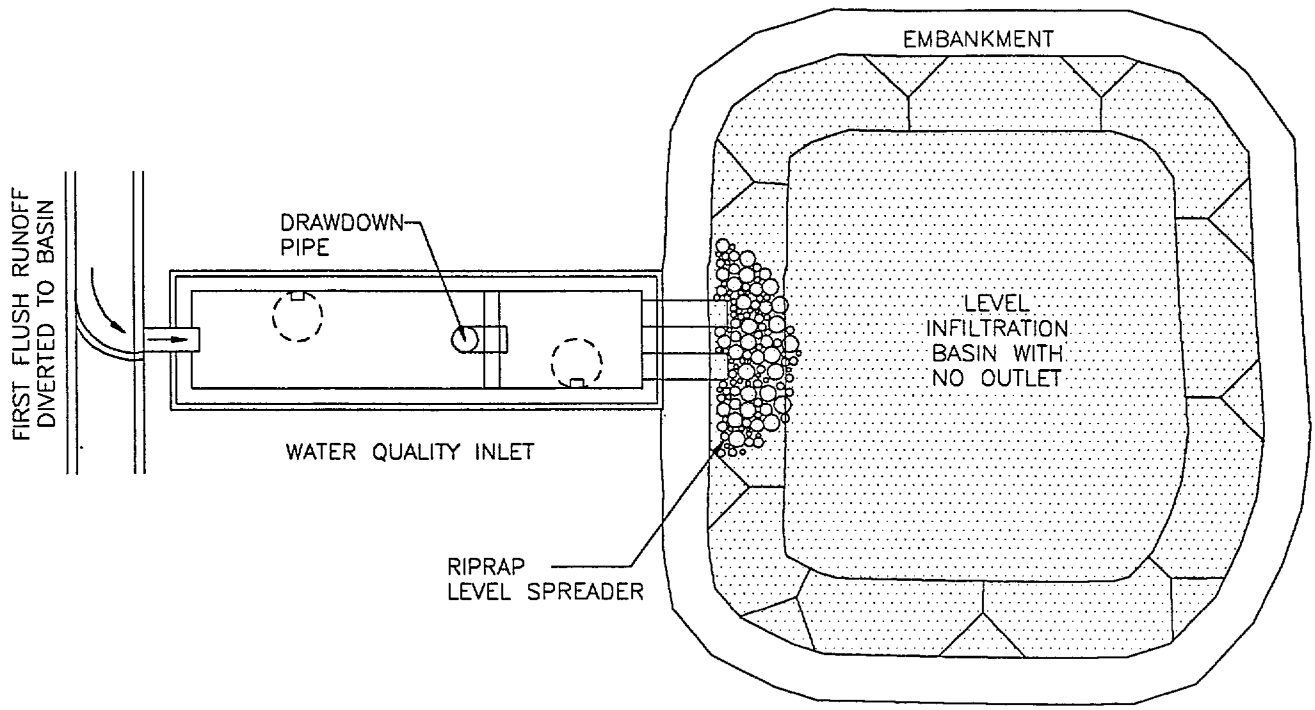
FIGURE 36



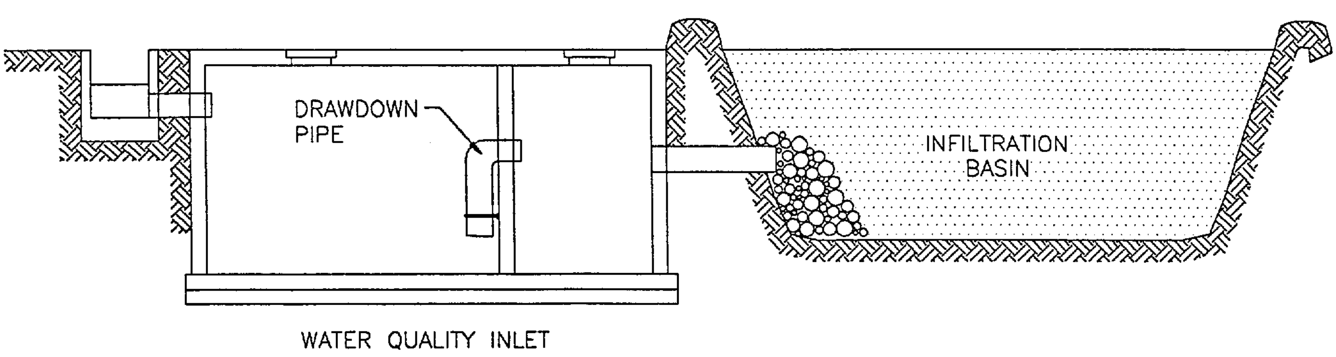
UNDERGROUND TRENCH WITH GRIT/OIL SEPARATOR

Smith Demer Normann Engineers – Planners – Surveyors – Landscape Architects Central Park Six Manhattan Square Suite 102 Hampton, Virginia 23666 (804)865-9610 (804)627-6900 Fax: (804)865-1533	HAMPTON ROADS PLANNING DISTRICT COMMISSION	REVS'D: CHK'D: DATE: REVS'D: CHK'D: DATE:	DRAWN: R.E.C. CHK'D: V.P.M. DATE: OCT. 1990 SCALE: N.T.S. PAGE 154

FIGURE 37



TOP VIEW



SIDE VIEW

OFF-LINE INFILTRATION BASIN

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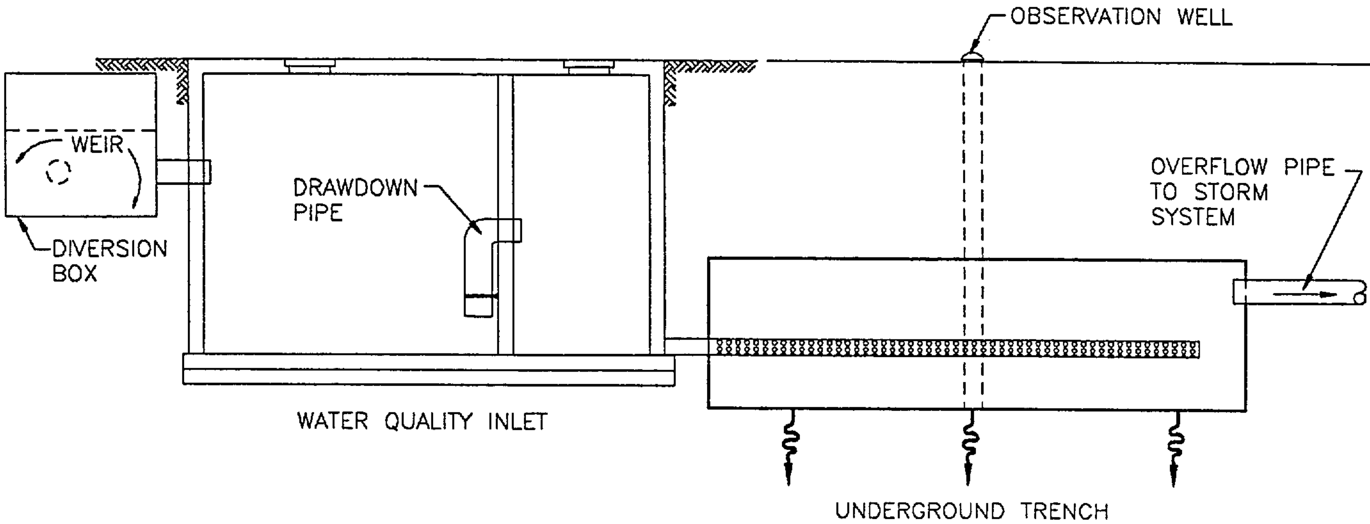
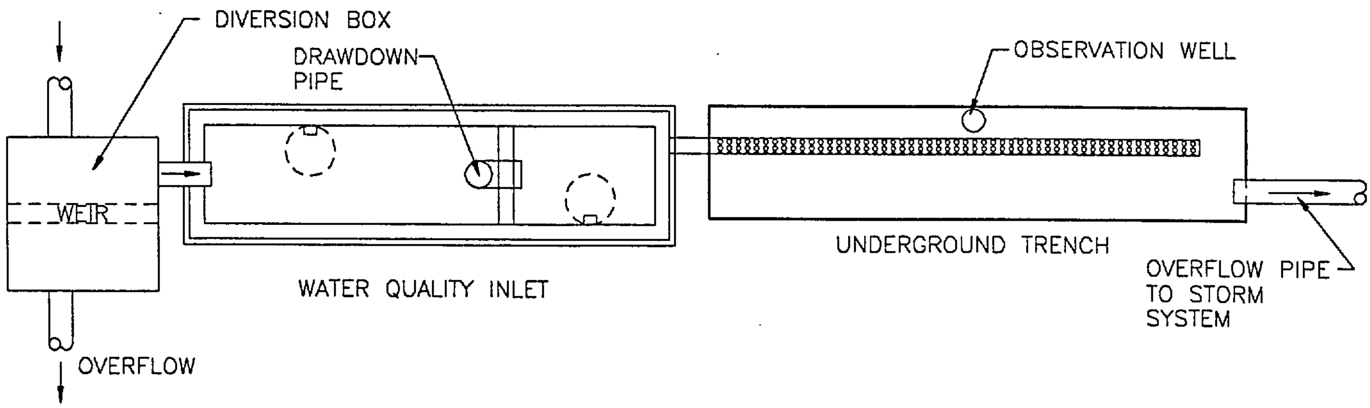
FIGURE 38



SIDE VIEW



TOP VIEW

OFF-LINE TRENCH SYSTEM

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TABLE II-5

SIZES OF COARSE AGGREATES — Open Graded

Va. Size No.	Amounts Finer Than Each Laboratory Sieve (Square Openings*), Percentage by Weight														
	4	3½	3	2½	2	1½	1	¾	½	3/8	No. 4	No. 8	No. 16	No. 50	No. 100
1	Min. 100	95±5		43±17		Max. 15		Max. 5							
2			Min. 100	95±5		43±17		Max. 15	Max. 5						
3					Min. 100	63±17		Max. 20	Max. 5						
357					Min. 100		60±20		20±10		Max. 5				
5						Min. 100	95±5	58±17	Max. 15	Max. 5					
56						Min. 100	95±5	58±17	25±10	Max. 15	Max. 5				
57						Min. 100	95±5		43±17		Max. 7	Max. 3			
68							Min. 100	95±5		48±17	Max. 20	Max. 8	Max. 5		
7								Min. 100	95±5	57±17	Max. 15	Max. 5			
78								Min. 100	95±5	60±20	Max. 20	Max. 8	Max. 5		
8									Min. 100	92±8	25±15	Max. 8	Max. 5		
9										Min. 100	92±8	25±15	Max. 10	Max. 5	
10										Min. 100	92±8				20±10

*In inches, except where otherwise indicated. Numbered sieves are those of the U.S. Standard Sieve Series.

Source: Virginia Department of Transportation - Road & Bridge Specifications
January 1987

Table 1.66b
SEEDING MIXTURES, RATES AND DATES: SOUTHERN PIEDMONT AND COASTAL PLAIN

SITE CONDITIONS	SEEDING MIXTURES	RATES		DATES		
		PER ACRE	PER 1000 ft^2	3/1 to 4/15	4/15 to 8/1	8/1 to 10/15
HIGH MAINTENANCE LAWNS	1. Tall fescue -------------------------- 90% Kentucky bluegrass------------------- 10%	250 lbs	6 lbs	X	no	X
LOW MAINTENANCE GENERAL USE	2. Tall fescue-------------------------- 50% Ladino clover------------------------ 10% Red clover--------------------------- 10% Korean Lespedeza--------------------- 15% Annual ryegrass---------------------- 15%	80 lbs	2 lbs	X	(a,b) X	X
	3. Tall fescue-------------------------- 50% Sericea lespedeza-------------------- 30% Annual ryegrass---------------------- 20%	70 lbs	1½ lbs	X	(a) X	X
DROUGHTY AREAS, SANDY SOILS	4. Tall fescue-------------------------- 50% Sericea lespedeza-------------------- 20% Korean lespedeza--------------------- 15% Annual ryegrass---------------------- 15%	80 lbs	2 lbs	X	(a,b) X	X
POORLY DRAINED AREAS	5. Tall fescue-------------------------- 65% Korean lespedeza--------------------- 20% Annual ryegrass---------------------- 10% Redtop-------------------------------- 5%	80 lbs	2 lbs	X	(a,b) X	X

a. After May 1, use 10 lb/A german millet or 2 lb/A weeping lovegrass in place of annual ryegrass.
b. After May 1, Korean lespedeza will not reseed itself. You may increase the amount of other legumes accordingly.

Source: Va SWCC

Source: Va SWCC

1980

Table 1.66c
CHARACTERISTICS OF GRASSES APPROPRIATE FOR EROSION CONTROL

COMMON NAME (BOTANICAL NAME)	LIFE CYCLE	SEASON	SOD-FORMER	BUNCH GRASS	GERMINATION TIME IN DAYS	pH RANGE	DRAINAGE TOLERANCE: DROUGHTY	WELL-DRAINED	MOD. WELL-DRAINED	SOMEWHAT POORLY DR.	POORLY DRAINED	FLOOD TOLERANT	MAINTENANCE REQUIREMENTS	REMARKS	SUGGESTED VARIETIES FOR VIRGINIA
KENTUCKY BLUEGRASS (Poa pratense)	P	C	X		10-28	6.0-7.0		X	X	X		X	Needs fertile soil, favorable moisture, and liberal phosphorus.	Suitable for fine turf. Poor drought and heat tolerance.	Many varieties
PERENNIAL RYEGRASS (Lolium perenne)	sP	C		X	5-14	5.5-7.5		X	X	X		X	Similar to bluegrass.	Best when used with bluegrass, as 20% or less of mixture. Quick germination.	Manhattan Citation Pennfine
RED FESCUE (Festucca rubra)	P	C	X		7-21	4.5-6.5	X	X	X	X			Do not fertilize heavily with nitrogen.	Include in fine turf. Shade and drought tolerant. Persists best in cool environments.	Pennlawn Jamestown
REED CANARYGRASS (Phalaris arundinacea)	P	C	X		5-21	5.0-7.5	X	X	X	X	X	X	Do not mow closely or often	Tall, coarse;adapted to wet soils, waterways, muck and peat soils.	Io
TALL FESCUE (festuca arundinacea)	P	C		X	5-14	5.5-8.0		X	X	X			Mow often to prevent bunchiness.	Widely adapted. Tolerates drought, infertility, moderate shade.	Kentucky 31
GERMAN MILLET (Setaria italica)	A	W		X	4-14	4.5-7.0	X	X	X	X			Do not use in fine turf.	Warm season temporary or companion grass.	No Named Varieties
ANNUAL RYEGRASS (Lolium multiflorum)	A	C		X	5-14	5.5-7.5		X	X	X		X	Do not use in fine turf.	Cool season temporary or companion grass. Cannot tolerate temperature extremes or drought. Somewhat shade tolerant.	No Named Varieties
OATS (Avena sativa)	A	C		X	5-10	5.5 7.0		X	X				Do not use in fine turf.	Cool season temporary or companion grass. Use spring oats.	Lang
REDTOP (Agrostis alba)	sP	C	X		5-10	4.0-7.5	X	X	X	X	X	X	Do not use in fine turf.	Cool season companion grass.Adapted to very acid, infertile soils. Can be used on smooth,steep slopes	No Named Varieties
RYE (Secale cereale)	A	C		X	4-7	5.5 7.0	X	X	X	X			Do not use in fine turf.	Cool season temporary or companion grass, best used in late fall seedings.	Abruzzi
WEEPING LOVEGRASS (Evagrostis curvula)	sP	W		X	5-14	4.5-8.0	X	X	X	X			Mow yearly to encourage persistence.	Warm season temporary or companion grass. Tolerates acid,infertile soils; steep,droughty slopes.	No Named Varieties

A-annual
P-perennial
sP-short-lived perennial, lasts 3-4 years

W-warm-season plant, grows in summer
C-cool-season plant, grows in spring and fall

Source: Va SWCC

1980

Table 1.66d
CHARACTERISTICS OF LEGUMES APPROPRIATE FOR EROSION CONTROL

COMMON NAME (Botanical Name)	LIFE CYCLE	SEASON	GERMINATION TIME IN DAYS	pH RANGE	DRAINAGE TOLERANCE: DROUGHTY	WELL-DRAINED	MOD. WELL-DRAINED	MOD. POORLY DRAINED	POORLY DRAINED	MAINTENANCE REQUIREMENTS	REMARKS	SUGGESTED VARIETIES FOR VIRGINIA
RED CLOVER (Trifolium pratense)	sP	C	7-21	6.0-7.0		X	X	X		Needs high phosphorus and potassium. Do not mow frequently.	Single plants, 10-18 inches tall; long tap roots. Useful with tall fescue in low-maintenance stands. Will reseed itself.	Kenstar Chesapeake Kenland Pennscott
WHITE CLOVER (Trifolium repens)	P	C	7-21	6.0-7.5		X	X			Needs favorable moisture, high fertility, high pH.	Prostrate plants spread by stolons. Cannot persist with tall plants.	Tillman Common White Dutch
CROWN VETCH (Coronilla varia)	P	C	14-21	5.5 8.3	X	X	X			Needs high lime or calcareous soil, high phosphorus. Will not persist under frequent mowing. Will not tolerate wet soil.	18-24" tall. Has spreading root stocks. Tolerates acid to pH 5.0 when soil has high lime content. Deep rooted, somewhat shade tolerant. Useful on steep slopes and rocky areas.	Chemung Penngift Emerald
FLATPEA (Lathyrus silvestrus)	P	C	14-28	5.0-7.0	X	X	X	X	X	Needs lime and high phosphorus. Do not mow closely.	Prostrate, spreading plants 2-3 ft. tall. Adapted to drought, low fertility, partial shade, cold winters. Chokes out woody vegetation.	Lathco
ANNUAL LESPEDEZAS (Lespedeza striata, L. stipulacea)	A	W	5-14	5.0-7.0	X	X	X	X	X	Do not mow closely.	Companion legume for warm seasons. Acid tolerant. Short tap roots. Will reseed itself.	Kobe
SERICEA LESPEDEZA (Lespedeza cuneata)	P	W	7-28	5.0-7.0	X	X	X	X		Will not persist under frequent mowing.	Very deep rooted. Drought tolerant. Useful on infertile slopes. Does not persist in coastal plain.	Serala Interstate

P-perennial
A-annual
sP-short-lived perennial, lasts 3-4 years

W-warm season plant, grows in summer
C-cool season plant, grows in spring and fall

NOTE: Seed of all legumes must be inoculated with the correct strain of bacteria.

APPENDIX III

GUIDANCE CALCULATION PROCEDURE

SOURCE: CHESAPEAKE BAY LOCAL ASSISTANCE DEPARTMENT.

INTRODUCTION

This procedure is designed to help applicants determine compliance with a locality's Chesapeake Bay Preservation Act program. This procedure does not supplant any information or requirement of other stormwater management programs, namely any local initiative adopted pursuant to either the Erosion and Sediment Control (ESC) Law [§ 10.1-560, et. seq.] or the Stormwater Management (SWM) Law [§ 10.1-603.1, et. seq.]. While all three programs are intended to protect water resources from further degradation, each requires separate engineering analysis. In general, these programs require calculations as follows:

- a CBPA program : stormwater quality
- a SWM program : stormwater quantity and quality
- an ESC program : two-year design storm runoff volumes and velocities

Many localities may combine all aspects into one, comprehensive program. This calculation procedure would then be just one aspect of that program and a development proposal's submittal.

STEP ONE: **Determine if the site is in a Chesapeake Bay Preservation Area.**

The Regulations[1] require localities to designate Chesapeake Bay Preservation Areas (CBPAs). Guidelines for local designation are contained in Chapters II and III of the *Local Assistance Manual* and Part III of the Regulations. CBPAs consist of two different classifications: Resource Protection Areas (RPAs) and Resource Management Areas (RMAs). The stormwater management criteria apply equally to both RPAs and RMAs.

While localities have flexibility to determine their own CBPAs, those areas will generally include the following land features:

In RPAs: tidal wetlands, nontidal wetlands contiguous to tidal wetlands, tidal shores, tributary streams, a buffer area (of not less than 100 feet), and other lands as designated by the locality;

In RMAs: floodplains, highly erodible soils, highly permeable soils, nontidal wetlands not in the RPA, and other land as designated by the locality.

Determine from the locality's designation maps and criteria if the site is subject to this procedure. Localities may require the entire site to comply with the Regulations even if only a portion of the site is in a CBPA. Determine the locality's requirement on total site compliance.

STEP TWO: **Determine if the site is classified as new development or redevelopment.**

The Regulations provide the following definitions:

Development means the construction, or substantial alteration of residential, commercial, industrial, institutional, recreational, transportation, or utility facilities or structures.

Redevelopment means the process of developing land that is or has been previously developed.

Check with the locality to see if further clarification is provided concerning redevelopment.

NOTE: **Any site in an Intensely Developed Area is automatically classified as redevelopment, regardless of the site's present or previous condition.**
[§ 3.4 of the Regulations]

For development, the post-development nonpoint source pollution runoff load cannot exceed the pre-development load based on "average land cover conditions." This standard can be referred to as a "no net increase" standard. STEP THREE will further discuss "average land cover conditions."

For redevelopment sites not served by BMPs, the post-development non-point source pollution runoff load must be 90 percent or less of the pre-development load for that site. This standard can be referred to as a "10 percent reduction" standard. Redevelopment criteria are not based on average land cover conditions.

For redevelopment sites with BMPs, the following provision(s) must be satisfied to constitute "being served by water quality best management practices":

(1) **In general, runoff pollution loads must have been calculated and the BMP selected for the expressed purpose of controlling NPS pollution.** However, if existing facilities can be shown to achieve the current standard of NPS pollution control, local authorities may consider the site as being served by water quality BMPs.

(2) **If BMPs are structural, facilities must currently be in good working order, performing at the design levels of service.** The local authority may require a review of both the original structural design and maintenance plans to verify this provision. A new maintenance agreement may be required to ensure consistency with the locality's SWM requirements.

STEP THREE: **Determine the relative pre-development pollutant load of the Keystone Pollutant (L_{pre}).**

The Keystone Pollutant for Tidewater Virginia is total phosphorous. The selection of total phosphorous as the keystone pollutant is discussed in Attachment A. For the remainder of this procedure, "pollutant" or "pollutant loading(s)" will mean total phosphorous.

Following development or redevelopment, impervious cover is the key determinant in the levels of pollutant export. Up to 90 percent of the atmospheric pollutants deposited on impervious surfaces are delivered to receiving waters.[2] So, for STEPS THREE and FOUR, the site designer need only determine the amount of total area subject to these criteria and the proposed amount of impervious cover (or equivalent). Guidance on determining equivalents is given in Attachment B. Worksheets A and B will help with these next two steps.

The zoning classification or proposed density of a site will allow applicants to estimate impervious cover. Compliance and final engineering calculations, however, should be based on impervious cover shown on the final site plan. Even so, localities and applicants are encouraged to "err" conservatively, as properties tend to become more impervious with time, e.g. the expansion of a structure, paving a driveway, adding more parking spaces. A conservative estimate indicates more, rather than less, impervious cover. Localities may wish to set a minimum for particular land uses but require the determination of proposed impervious cover and **use the higher number.** Representative land use categories and associated pollutant exports are shown in Table 1.

FOR DEVELOPMENT:

Average Land Cover Conditions ($I_{watershed}$)

Just as a locality must designate CBPAs, a locality must also establish baseloads for watersheds within its jurisdiction. Once set, the baseload will not change unless technology provides a more precise answer. Watershed delineations serve as the baseline for a calculation procedure and do not constitute an additional regulatory step. The two options available to localities are:

1. A locality will designate watersheds within its jurisdiction and calculate the <u>average</u> total phosphorus loading and equivalent impervious cover for each individual watershed, or

2. A locality will declare its entire jurisdiction as part of Virginia's Chesapeake Bay watershed with an <u>average</u> total phosphorus loading (F_{VA}) of **0.45 pounds/acre/year** and an equivalent impervious cover (I_{VA}) of 16 percent.

Some localities may begin with OPTION TWO while they gather the necessary data for OPTION ONE. Guidance on how a locality should calculate individual watershed loads is provided in Attachment B. Discussion of the default loadings is in Attachment C.

With $I_{watershed}$, L_{pre} can be calculated using the Simple Method.[3] The derivation of the Simple Method can be found in Appendix A of *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs,* published by the Metropolitan Washington Council of Governments.

$$\boxed{L_{pre} = P \times P_j \times [0.05 + 0.009(I_{watershed})] \times C \times A \times 2.72 / 12}$$

where:

L_{pre} = relative pre-development total phosphorus load (in lbs/yr)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storm with no runoff = 0.9
$I_{watershed}$ = equivalent impervious cover for watershed, or "average land cover conditions" (percent expressed in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)
= 0.26 mg/l when $I_{watershed} < 20$
= 1.06 mg/l when $I_{watershed} \geq 20$
A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion factors

FOR REDEVELOPMENT:

Pre-development loads for redevelopment sites are <u>not</u> based on average land cover conditions. Instead, pre-development loads are based on the site conditions at the time of plan submittal. Therefore, determine <u>existing</u> impervious cover or equivalent.

With $I_{site(pre)}$, L_{pre} can be calculated using the Simple Method.

$$L_{pre} = P \times P_j \times [0.05 + 0.009(I_{site(pre)})] \times C \times A \times 2.72 / 12$$

where:

L_{pre} = relative pre-development total phosphorus load (in lbs)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storm with no runoff = 0.9
$I_{site(pre)}$ = equivalent pre-development impervious cover of the site (percent expressed in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)
= 0.26 mg/l when $I_{site(pre)} < 20$
= 1.06 mg/l when $I_{site(pre)} \geq 20$
A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion numbers

STEP FOUR: **Determine the relative post-development pollutant load (L_{post}).**

Just as with STEP THREE, the designer needs to know the post-development impervious cover (or equivalent). For both new development and redevelopment, post-development loadings are site-specific.

FOR NEW DEVELOPMENT

Again, the Simple Method is used.

$$L_{post} = P \times P_j \times [0.05 + 0.009(I_{site(post)})] \times C \times A \times 2.72 / 12$$

where:

L_{post} = relative post-development total phosphorus load (in lbs)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storms with no runoff = 0.9

$I_{site(post)}$ = equivalent post-development impervious cover (percent in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)

- For Option One: Locally Designated Watersheds
  = 0.26 mg/l when $I_{site(post)} < 20$
  = 1.06 mg/l when $I_{site(post)} \geq 20$
- For Option Two: Va. Chesapeake Bay Default
  =0.26 mg/l for all $I_{site(post)}$

A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion factors

FOR REDEVELOPMENT:

Again, the Simple Method is used.

$$L_{post} = P \times P_j \times [0.05 + 0.009(I_{site(post)})] \times C \times A \times 2.72 / 12$$

where:

L_{post} = relative post-development total phosphorus load (in lbs)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storms with no runoff = 0.9
$I_{site(post)}$ = equivalent post-development impervious cover (percent in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)
= 0.26 mg/l when $I_{site(post)} < 20$
= 1.06 mg/l when $I_{site(post)} \geq 20$
A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion factors

STEP FIVE: **Determine the relative removal requirements (RR).**

Remember from STEP TWO, the performance standards are different.

FOR DEVELOPMENT:

$$RR = L_{post} - L_{pre}$$

FOR REDEVELOPMENT:

$$RR = L_{post} - 0.9(L_{pre})$$

If the calculated number is less than or equal to zero, STOP. ***Note that in watersheds using the Tidewater weighted average, F_{VA} = 0.45 lbs/ac/yr, new single-family home parcels one acre or greater do not require BMPs.***

If no BMPS are required, the applicant need only submit documentation to support his or her findings. If such findings are found correct by local officials, the applicant has then satisfied the stormwater management criteria. The state Stormwater Management Law and the Erosion and Sediment Control Law also deal with other water resource related provisions, such as quantity-related requirements.

If removal efficiencies are required, continue on with STEP SIX.

STEP SIX: **Identify BMP options for the site.**

Best Management Practices (BMPs) can be used to remove pollutants. BMPs are not always structural. For instance, trash removal can drastically reduce the amount of solid wastes that reach our streams. However, for the purpose of this discussion BMPs will mean any structural or mechanical device capable of preventing or reducing the amount of pollution from nonpoint sources.

The use of certain BMPs may be limited on some sites by soils, topography, area and other physical characteristics. Most BMPs can only be applied under restricted site conditions. Improperly sited, a BMP cannot perform as designed and may become a chronic maintenance problem. A poorly maintained BMP may even contribute pollutants, e.g. an eroding pond embankment sends sediment into the receiving stream.

BMPs and their associated pollutant removal efficiencies are shown in Table 2. This list is by no means a complete listing of available BMPs, nor does appearance on this list indicate appropriateness for a given situation.

STEP SEVEN: **Determine if feasible BMP options can meet the pollutant removal requirement.**

If runoff from the entire site passes through the BMP, the applicant need only select a BMP with an efficiency rating equal to or greater than the efficiency required [as determined in STEP FIVE]. If, as is usually the case, only portions of the site are covered by BMPs, a weighted summation must be made.

Localities may allow pollutant reduction credits for serving off-site areas which drain through BMPs on the subject site. However, while applicants might claim pollutant reduction credits for serving off-site areas, applicants MAY NOT claim credit for one or more off-site BMPs serving their property (even if, in fact, they do). Neither the Act nor the Regulations allow for such an off-set program.

Worksheet C will help with this step of the procedure.

If no combination of BMPs can meet the required standard, the applicant must consider a different site design. Increasing the proportion of site area covered with vegetation is one of the best ways of lowering the required removal efficiencies. A different site layout may make a more appropriate BMP possible; for example, placing structures on "tight" soils may leave more permeable soil for infiltration areas.

ENDNOTES

[1] Chesapeake Bay Local Assistance Board, Final Regulations: VR 173-02-01 *Chesapeake Bay Preservation Area Designation and Management Regulations.* September 1989.

[2] Thomas R. Schueler, ***Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*** (Washington, D.C.: Metropolitan Washington Council of Government, Department of Environmental Programs, 1987), 1.4.

[3] Ibid, 1.9-1.13.

ANNUAL STORM PHOSPHOROUS EXPORT — TABLE 1

For Existing Urban Land Uses
(in pounds/acre/year)

LAND USES	IMPERVIOUS COVER (%)	ANNUAL RAINFALL (in) 40	41	42	43	44	45
	0	0.11	0.11	0.11	0.11	0.12	0.12
5.0 acre residential lots	5	0.20	0.21	0.21	0.22	0.22	0.23
2.0 acre residential lots	10	0.30	0.30	0.31	0.32	0.33	0.33
1.0 acre residential lots	15	0.39	0.40	0.41	0.42	0.43	0.44
	16	0.41	0.42	0.43	0.44	0.45	0.46
	17	0.43	0.44	0.45	0.46	0.47	0.48
	18	0.45	0.46	0.47	0.48	0.49	0.51
	19	0.47	0.48	0.49	0.50	0.52	0.53
0.50 acre residential lots	20	2.03	2.08	2.13	2.18	2.23	2.28
0.33 acre residential lots	25	2.42	2.48	2.54	2.61	2.67	2.72
0.25 acre residential lots	30	2.82	2.89	2.96	3.03	3.10	3.17
	35	3.22	3.30	3.38	3.46	3.54	3.62
Townhouses	40	3.61	3.70	3.79	3.88	3.97	4.06
	45	4.01	4.11	4.21	4.31	4.41	4.51
	50	4.41	4.52	4.63	4.74	4.85	4.96
Garden Apartments	55	4.80	4.92	5.04	5.16	5.28	5.40
	60	5.20	5.33	5.46	5.59	5.72	5.85
	65	5.60	5.74	5.88	6.02	6.16	6.30
Light	70	5.99	6.14	6.29	6.44	6.59	6.74
Commercial/Industrial	75	6.39	6.55	6.71	6.87	7.03	7.19
	80	6.79	6.96	7.13	7.29	7.46	7.63
Heavy	85	7.98	8.17	8.37	8.57	8.77	8.97
Commercial/Industrial	90	7.58	7.77	7.96	8.15	8.34	8.53
	95	7.98	8.17	8.37	8.57	8.77	8.97
Asphalt/Pavement	100	8.37	8.58	8.79	9.00	9.21	9.42

For Non-Urban Land Uses
(in pounds/acre/year)

LAND USE	SILT LOAM SOILS	LOAM SOILS	SANDY LOAM SOILS
Conventional Tillage Cropland	3.71	2.42	0.83
Conservation Tillage Cropland	2.32	1.52	0.52
Pasture Land	0.91	0.59	0.20
Forest Land	0.19	0.12	0.04

STRUCTURAL BMPS FOR CHESAPEAKE BAY PRESERVATION AREAS — *TABLE 2*

	Acceptable BMP	Average Total P Removal Efficiency
A.	Extended Detention	
	(1) Design 2 (6-12):	20%
	(2) Design 3 (24 hours):	30%
	(3) Design 4 (shallow marsh):	50%
B.	Wet Pond	
	(1) Design 5 (0.5 in/imp.ac):	35%
	(2) Design 6 (2.5 V_r):	40-45%
	(3) Design 7 (4.0 V_r):	50%
C.	Infiltration	
	(1) Design 8 (0.5 in/imp. ac):	50%
	(2) Design 9 (1.0 in/imp. ac):	65%
	(3) Design 10 (2-year storm):	70%
D.	Grassed Swale	
	(1) Design 15 (check dams):	10-20%

These designs are taken from Metropolitan Washington Council of Governments, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, ,1987

Effeciency ratings are taken from John P. Hartigan, P.E., *Three Step Process for Evaluating Compliance with BMP Requirements for Chesapeake Bay Preservation Areas*, 1990

WORKSHEET A : NEW DEVELOPMENT *OPTION ONE: LOCALLY DESIGNATED WATERSHEDS*

1 Compile site-specific data and determine site imperviousness (I_{site}).

		POST-DEVELOPMENT
A*		= ________ acres
I_a:**	structures	= ________ acres
	parking lot	= ________ acres
	roadway	= ________ acres
	other	= ________ acres
		= ________ acres
		= ________ acres
	total I_a	= ________ acres
I_{site} = (total I_a/A) × 100		= ________ (percent expressed in whole numbers)

* Although the area subject to regulations may be only the area actually in a CBPA, some localities may require all of the site to comply with criteria.

** I_a represents the actual amount of impervious area.

2 Determine the average land cover conditions ($I_{watershed}$).

Use $I_{watershed}$ as determined by the locality. If $I_{watershed} < 20$, use C_{pre} = 0.26mg/l. If $I_{watershed} \geq 20$, use C_{pre} = 1.08 mg/l.

3 Determine need to continue.

I_{site} = ________ % (from Step 1)

$I_{watershed}$ = ________ % (from Step 2)

If $I_{site} \leq I_{watershed}$, STOP and submit analysis to this point.

If $I_{site} > I_{watershed}$, CONTINUE.

4 Set constants.

P_j = unitless rainfall correction factor
= 0.9 for all of Tidewater Virginia

C_{post} = flow weighted mean concentration of total phosphorus
= 0.26 mg/l for $I_{site} < 20$
= 1.08 mg/l for $I_{site} \geq 20$.

P = annual rainfall depth in inches
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolian area
= 45 inches for Hampton Roads area

12 and 2.72 are used in the equation as unit conversion factors.

WORKSHEET A : NEW DEVELOPMENT *Option One: Locally Designated Watersheds*

5 **Calculate the pre-development load (L_{pre}).**

L_{pre} $= P \times P_j \times [0.05 + (0.009 \times I_{watershed})] \times C_{pre} \times A \times 2.72 / 12$

$= \_\_\_\_ \times 0.9 \times [0.05 + (0.009 \times \_\_\_\_)] \times \_\_\_\_ \times \_\_\_\_ \times 2.72 / 12$

$= \_\_\_\_\_\_\_\_$ pounds per year

6 **Calculate the post-development load (L_{post}).**

L_{post} $= P \times P_j \times [0.05 + (0.009 \times I_{site})] \times C_{post} \times A \times 2.72 / 12$

$= \_\_\_\_ \times 0.9 \times [0.05 + (0.009 \times \_\_\_\_)] \times \_\_\_\_ \times \_\_\_\_ \times 2.72 / 12$

$= \_\_\_\_\_\_\_\_$ pounds per year

7 **Calculate the pollutant removal requirement (RR).**

RR $= L_{post} - L_{pre}$

$= \_\_\_\_\_\_\_\_ - \_\_\_\_\_\_\_\_$

$= \_\_\_\_\_\_\_\_$ pounds per year

To determine the overall BMP efficiency required (%RR) when selecting BMP options:

%RR $= RR / L_{post} \times 100$

$= (\_\_\_\_ / \_\_\_\_) \times 100$

$= \_\_\_\_\_\_\_\_$ %

WORKSHEET A : NEW DEVELOPMENT

OPTION TWO: VA. CHESAPEAKE BAY DEFAULT

1 Compile site-specific data and determine site imperviousness (I_{site}).

			POST-DEVELOPMENT
A*		=	________ acres
I_a:**	structures	=	________ acres
	parking lot	=	________ acres
	roadway	=	________ acres
	other	=	________ acres
		=	________ acres
		=	________ acres
	total I_a	=	________ acres
I_{site} = (total I_a/A) × 100		=	________ (percent expressed in whole numbers)

* Although the area subject to regulations may be only the area actually in a CBPA, some localities may require all of the site to comply with criteria.
** I_a represents the actual amount of impervious area.

2 Determine the average land cover conditions ($I_{watershed}$).

Use $I_{watershed} = I_{VA} = 16$ because $F_{average} = 0.45$ lbs/ac/yr for Virginia's Chesapeake Bay Watershed. Use $C_{pre} = 0.26$ mg/l.

3 Determine need to continue.

I_{site} = ________ % (from Step 1)
$I_{watershed}$ = 16 % (from Step 2)

If $I_{site} \leq I_{watershed}$, STOP and submit analysis to this point.
If $I_{site} > I_{watershed}$, CONTINUE.

4 Set constants.

P_j = unitless rainfall correction factor
= 0.9 for all of Tidewater Virginia

C = flow weighted mean concentration of total phosphorus
= 0.26 mg/l for all I_{site}

P = annual rainfall depth in inches
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolian area
= 45 inches for Hampton Roads area

12 and 2.72 are used in the equation as unit conversion factors.

WORKSHEET A : NEW DEVELOPMENT

OPTION TWO: VA. CHESAPEAKE BAY DEFAULT

5 Calculate the pre-development load (L_{pre}).

L_{pre} = $P \times P_j \times [0.05 + (0.009 \times I_{watershed})] \times C_{pre} \times A \times 2.72 / 12$

= ____ $\times 0.9 \times [0.05 + (0.009 \times$ ____$)] \times 0.26 \times$ ____ $\times 2.72 / 12$

= ________ pounds per year

6 Calculate the post-development load (L_{post}).

L_{post} = $P \times P_j \times [0.05 + (0.009 \times I_{site})] \times C \times A \times 2.72 / 12$

= ____ $\times 0.9 \times [0.05 + (0.009 \times$ ____$)] \times 0.26 \times$ ____ $\times 2.72 / 12$

= ________ pounds per year

7 Calculate the pollutant removal requirement (RR).

RR = $L_{post} - L_{pre}$

= ________ - ________

= ________ pounds per year

To determine the overall BMP efficiency required (%RR) when selecting BMP options:

%RR = RR / $L_{post} \times 100$

= (____ / ____) $\times 100$

= ________ %

WORKSHEET B : REDEVELOPMENT

1 Compile site-specific data.

		PRE-DEVELOPMENT	POST-DEVELOPMENT
A*		= _______ acres	= _______ acres
I_a:	structures	= _______ acres	= _______ acres
	parking lot	= _______ acres	= _______ acres
	roadway	= _______ acres	= _______ acres
	other	= _______ acres	= _______ acres
		= _______ acres	= _______ acres
		= _______ acres	= _______ acres
	total I_a	= _______ acres	= _______ acres
$I = (\text{total } I_a / A) \times 100$		= _______ percent expressed in whole numbers	= _______ percent expressed in whole numbers
$R_v = 0.05 + (0.009 \times I)$		= _______ unitless	= _______ unitless
C:	$I \geq 20 = 1.08$ mg/l $I < 20 = 0.26$ mg/l	= _______ mg/l	= _______ mg/l

* Although the area subject to regulations may be only the area actually in a CBPA, some localities may require all of the site to comply with criteria.

2 Set constants.

P_j = unitless rainfall correction factor
= 0.9 for all of Tidewater Virginia

P = annual rainfall depth in inches
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area

12 and 2.72 are used in the equation as unit conversion factors.

3 Calculate the pre-development load (L_{pre}).

$L_{pre} = P \times P_j \times R_{v(pre)} \times C_{pre} \times A \times 2.72 / 12$

= _____ × 0.9 × _____ × _____ × _____ × 2.72 / 12

= _________ pounds per year

4 Calculate the post-development load (L_{post}).

$L_{post} = P \times P_j \times R_{v(post)} \times C_{post} \times A \times 2.72 / 12$

= _____ × 0.9 × _____ × _____ × _____ × 2.72 /12

= _________ pounds per year

5 Calculate the pollutant removal requirement (RR).

RR $= L_{post} - (0.9 \times L_{pre})$

= _________ - (0.9 × _________)

= _________ pounds per year

%RR $= (RR / L_{post}) \times 100$

= (_____ / _____) × 100

= _________ %

WORKSHEET C: COMPLIANCE

Select BMP options using screening tools and list them below. Then calculate the load removed for each option. DO NOT LIST BMPs IN SERIES HERE.

1

Selected Option	Removal Efficiency (%/100)	×	Fraction of CBPA Drainage Area Served (expressed in decimal form)	×	L_{post} (lbs/yr)	=	Load Removed (lbs/yr)
______	______		______		______		______
______	______		______		______		______
______	______		______		______		______

2a **Estimate parameters for non-CBPA drainage areas on the project site (if the locality does not require complete compliance for the whole site). If the locality requires compliance for the whole site, omit this step.**

A (on site, non-CBPA)		= ______	acres
I_a:	structures	= ______	acres
	parking lot	= ______	acres
	roadway	= ______	acres
	other	= ______	acres
		= ______	acres
		= ______	acres
	total I	= ______	acres

$I = (\text{total } I_a/A) \times 100$ = ______ %

$R_v = 0.05 + (0.009 \times I)$ = ______

C: $I \geq 20 = 1.08$ mg/l = ______ mg/l
$I < 20 = 0.26$ mg/l

When using VIRGINIA CHESAPEAKE BAY DEFAULT ($F_{V_a} = 0.45$ lbs/ac/yr), C=0.26 mg/l for all I_{site}.

2b **Calculate post-development load for on-site non-CBPAs.**

$L_{post(outside)}$ $= P \times P_j \times R_v \times C \times A \times 2.72/12$

$= \_\_\_\_ \times 0.9 \times \_\_\_\_ \times \_\_\_\_ \times \_\_\_\_ \times 2.72 / 12$

= ________ pounds per year

Revised 7/90

3 **Determine loadings for off-site areas if the locality allows this option.**

$I_{watershed}$ = from locality <u>OR</u> $I_{watershed} = I_{VA} = 16$

If $I_{watershed} < 20$, use $C_{offsite} = 0.26$ mg/l.
If $I_{watershed} \geq 20$, use $C_{offsite} = 1.08$ mg/l.
If $I_{watershed} = I_{VA}$ use $C_{offsite} = 0.26$mg/l.

$L_{offsite}$ $= P \times Pj \times [0.05 + (0.009 \times I_{watershed})] \times C_{offsite} \times A_{offsite} \times 2.72 / 12$

$= \_\_\_\_ \times 0.9 \times [0.05 + (0.009 \times \_\_\_\_)] \times \_\_\_\_ \times \_\_\_\_ \times 2.72 / 12$

= ________ pounds per year

4 **Total non-CBPA pollutant loading.**

Step 3 + Step 4 = total non-CBPA loading

________ + ________ = ________ pounds per year

5 **Calculate credits if the locality allows this option.**

Selected Option	Removal Efficiency (%/100)	×	L_{post} (lbs/yr)	=	Load Removed (lbs/yr)
________	________		________		________
________	________		________		________
________	________		________		________

6 **Calculate overall compliance.**

Step 1 + Step 5 = total load removed

________ + ________ = ________ pounds per year

If total load removed > removal requirement, criteria are satisfied.

________ > ________

Revised 7/90

ATTACHMENT A

Many different pollutants can be identified in our streams and water bodies. The Regulations merely require the control of "nonpoint source (nps) pollution." The Model Ordinance defines NPS as pollution consisting of constituents such as sediment, nutrients, and organic and toxic substances from diffuse sources. Trying to deal with all the possible pollutants would make any calculation procedure complicated and expensive. To simplify the calculations needed, a "keystone" pollutant can be selected. A keystone pollutant shares the general characteristics of most other pollutants. By removing the keystone pollutant, other important pollutants will be simultaneously removed. Chapter 2 of *A Framework for Evaluating Compliance with the 10% Rule*[1] reviews each of the major pollutants found in urban runoff for their suitability as the keystone pollutant, based on the following three criteria:

1. The pollutant must have a well-defined adverse impact on the Chesapeake Bay.

2. The pollutant should exist in a "composite" form, i.e. in a roughly equal split between particulate and soluble phases.

3. Enough research data must be available to provide a reasonable basis for estimating how keystone pollutant loads change in response to development and to current stormwater control measures.

The only urban pollutants that appear to meet all three criteria for suitability as a keystone pollutant are: total phosphorus, total nitrogen and zinc (Table 3). Of these three, total phosphorus exists in the most equivalent proportions of soluble and particulate forms (40/60). Total nitrogen and zinc are less proportionate, at 20/80 and 25/75, respectively.

TABLE 3

Pollutant	Well-Defined Impacts on the Bay?	Composite Form?	Adequate Data?
Sediment	yes	no	no
Total Phosphorous	yes	yes	yes
Total Nitrogen	yes	yes	yes
Coliform Bacteria	yes	no	no
BOD/COD	yes	yes	no
Oil/Grease	yes	no	no
Zinc	yes	yes	yes
Lead	yes	no	yes
Toxics	no	no	no

By removing total phosphorus, an equal or greater level of removal for most other urban pollutants is simultaneously obtained. An equal or higher level or removal is possible for nearly every other pollutant, except total nitrogen. Total nitrogen is primarily found in soluble form, which is much more difficult to remove with current techniques. Nevertheless, by removing phosphorus, a reasonable degree of nitrogen is still removed as well.

Based on this review, total phosphorus was selected as the best candidate for the keystone pollutant in Tidewater Virginia. In doing so, Virginia will target the same pollutant as Maryland, preserving some consistency in our multi-state Bay preservation effort.

ENDNOTE:

[1] Schueler, Thomas R. and Matthew R. Bley, ***A Framework for Evaluating Compliance with the Chesapeake Bay Critical Area*** (Washington, D.C.: Maryland Critical Area Commission and Maryland Department of the Environment, 1987).

ATTACHMENT B

The Regulations require new development stormwater management criteria be based on "average land cover conditions." Watershed designations serve as the baseline for a calculation procedure and do not constitute an additional regulatory step. Localities will have two options:

1. A locality will designate watersheds within its jurisdiction and calculate the average phosphorus loading and impervious cover for each individual watershed, or

2. A locality will declare its entire watershed as part of Virginia's Chesapeake Bay watershed with an average phosphorus loading of 0.45 pounds/acre/year and impervious cover of 16 percent.

A locality may begin with Option Two while they gather the necessary data for Option One. Figure 1 shows how Fairfax County could break up its watersheds. This discussion revolves around Option One. Option Two is discussed in Attachment C.

To determine average land cover conditions within a watershed, the locality must follow a three-step procedure:

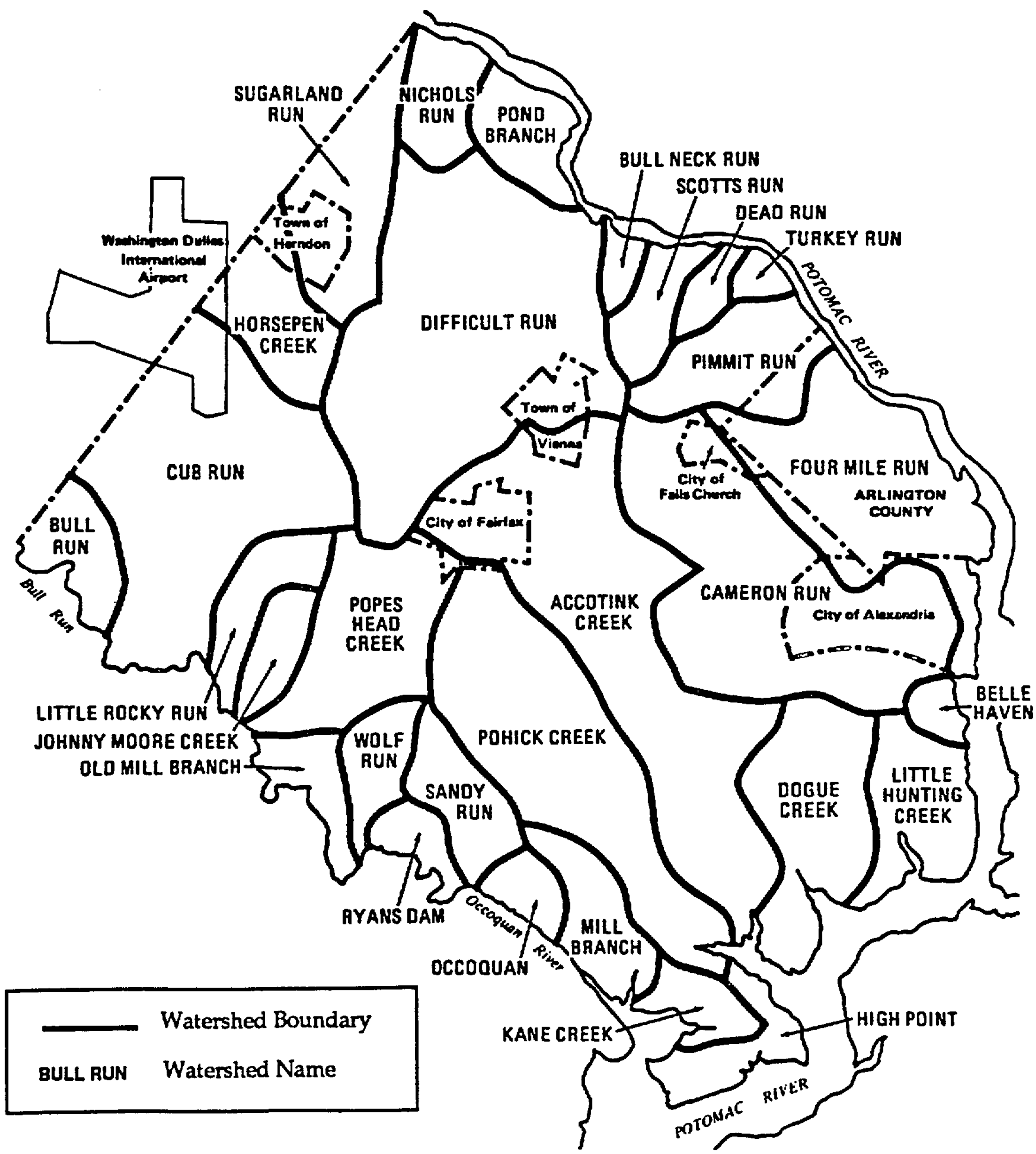
1. **Evaluate individual watersheds.** We recommend a minimum watershed area of 100 acres. Localities may wish however, to use watershed delineations used for other aspects of its work, e.g. a sanitary sewer master plan.

2. **Know existing land use data.** The Regulations are based on present land uses, not proposed land uses. A comprehensive plan is more future oriented than a zoning map. Still, a zoning map does not always indicate present use. A locality may also be able to use current aerial photographs. Data may be cross-referenced with Commissioner of Revenue information.

3. **Compute a weighted average of impervious cover (or its equivalent).** The Simple Method (and the nonpoint source pollution load) is highly dependent on the percent of impervious cover. Some land uses contribute nonpoint source pollution but do not have "impervious covers," e.g. forest and agriculture lands. Therefore, conversions, or equivalents, must be determined. Use Table 1 to find equivalent loading/impervious factors for non-urban uses. Localities may use other documented loading factors, especially if found to be more appropriate to that locality, as long as the factors are used consistently.

   Weighted averages are frequently computed for quantity related analyses and this process is identical. Figure 2 shows how average land cover conditions might be calculated for a 100-acre watershed.

POSSIBLE FAIRFAX COUNTY WATERSHEDS — *FIGURE 1*

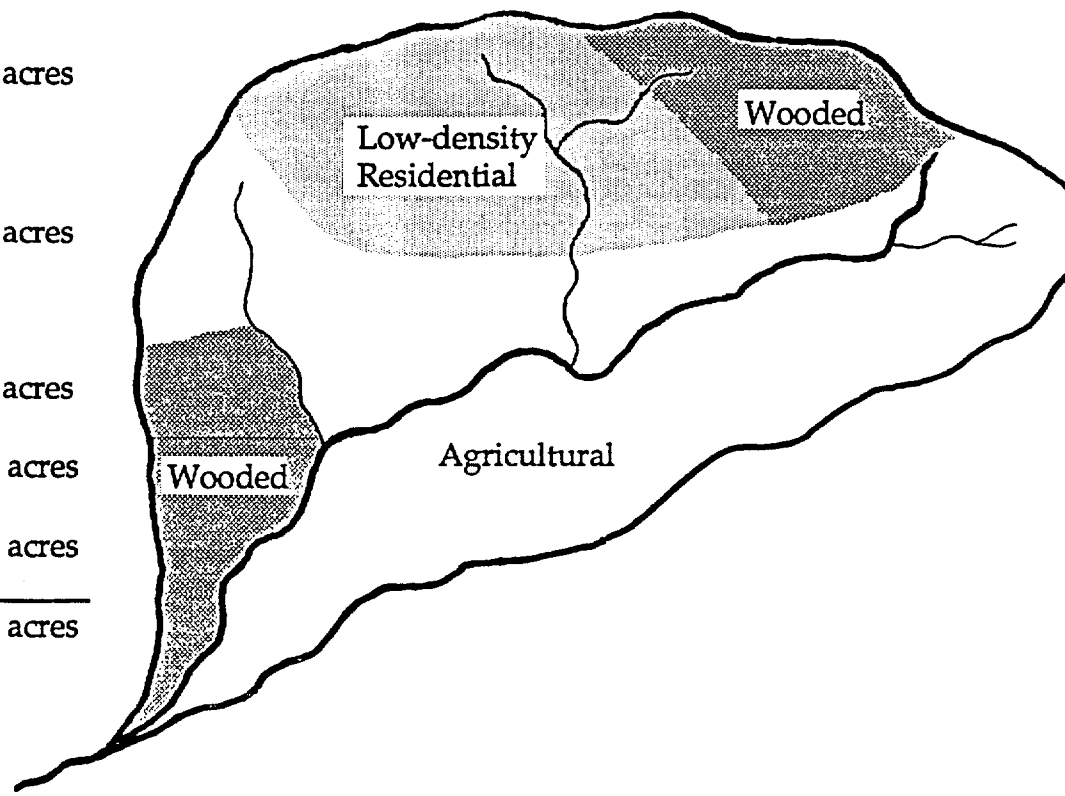


Source: County of Fairfax, *1987 Annual Report on the Environment* (Fairfax, Va.: Environmental Quality Advisory Council and Office of Comprehensive Planning, 1987), p. 16

CALCULATING AVERAGE LAND COVER CONDITIONS *FIGURE 2*

100 acre Watershed

Wooded	= 20 acres
Low-density Residential (1-acre lots)	= 20 acres
Agriculture	
Pasture	= 30 acres
Conservation tillage	= 15 acres
Conventional tillage	= 15 acres
Total acreage	100 acres



Land Use	Loading: * lbs/acre/year	# of Acres	Weighted Load: lbs/year
Wooded	0.12	20	2.4
1-acre lots	0.42	20	8.4
Pasture	0.59	30	17.7
Conventional	2.42	15	36.3
Conservation	1.52	15	22.8
		100	87.6

* Phosphorous; based on rainfall of P=43 inches/year and loam soils.

$$\Sigma = \frac{\text{Sum of weighted loadings}}{\text{total acreage}}$$

$$= \frac{0.12(20) + 0.42(20) + 0.59(30) + 2.42(15) + 1.52(15)}{20 + 20 + 30 + 15 + 15} = \frac{88 \text{ lbs per year}}{100 \text{ acres}} = 0.88 \text{ lbs per acre per year}$$

$$\text{Equivalent Impervious Cover} = I_{watershed} = 19$$

ATTACHMENT C

Not all localities will have the ability to designate individual watersheds and compute an average watershed baseload. For that reason, the department has determined a default load for Tidewater Virginia.

Following the procedure outlined in Attachment B:

1. Designate watershed.

   The department chose the entire Virginia portion of the Chesapeake Bay watershed – not just Tidewater Virginia (as defined by the Chesapeake Bay Preservation Act). The department encourages multi-jurisdictional cooperation among localities to designate large-scale watersheds as well.

2. Evaluate existing land use data.

   Existing land use data is given in ***Virginia's Chesapeake Bay Initiatives: First Annual Progress Report*** (September 1985) produced by the Virginia Council on the Environment. This breakdown is shown in Figure 3.

3. Compute a weighted average of impervious cover (or its equivalent).

   Because urban areas are most likely to adopt Option One, urban areas are excluded from the weighted average. In addition, loading rates for "urban" areas are highly variable.

F_{VA} = relative total phosphorus load for Virginia's Chesapeake Bay watershed

$F_{(X)}$ = relative total phosphorus load for any land use (X)

$$F_{VA} = \%FOR(F_{FOR}) + \%PAST(F_{PAST}) + \%CST(F_{CST}) + \%CVT(F_{CVT})$$

$$= 0.66(0.12) + 0.21(0.59) + 0.07(1.52) + 0.06(2.42)$$

$$= 0.45 \text{ lbs/ac/yr}$$

Use Table 1 to determine the equivalent impervious cover. The average loading, F_{VA} = 0.45 lbs/ac/yr, falls between impervious covers of 16 to 18 percents. Because of the differing annual rainfall across the state, the department has choosen the most conservative value of 16.

$$F_{VA} = 0.45 \text{ lb/ac/yr} \iff I_{VA} = 16\%$$

Therefore, the default load for Virginia's Chesapeake Bay watershed is 0.45 lb/ac/yr with an equivalent impervious cover of 16 percent. Localities are encouraged, but not required, to customize this aspect of the procedure, even if computing individual watersheds is not feasible. The Town of Herndon might use $I_{VA} = 18$, Caroline County might use $I_{VA} = 17$ and Isle of Wight County would retain $I_{VA} = 16$.

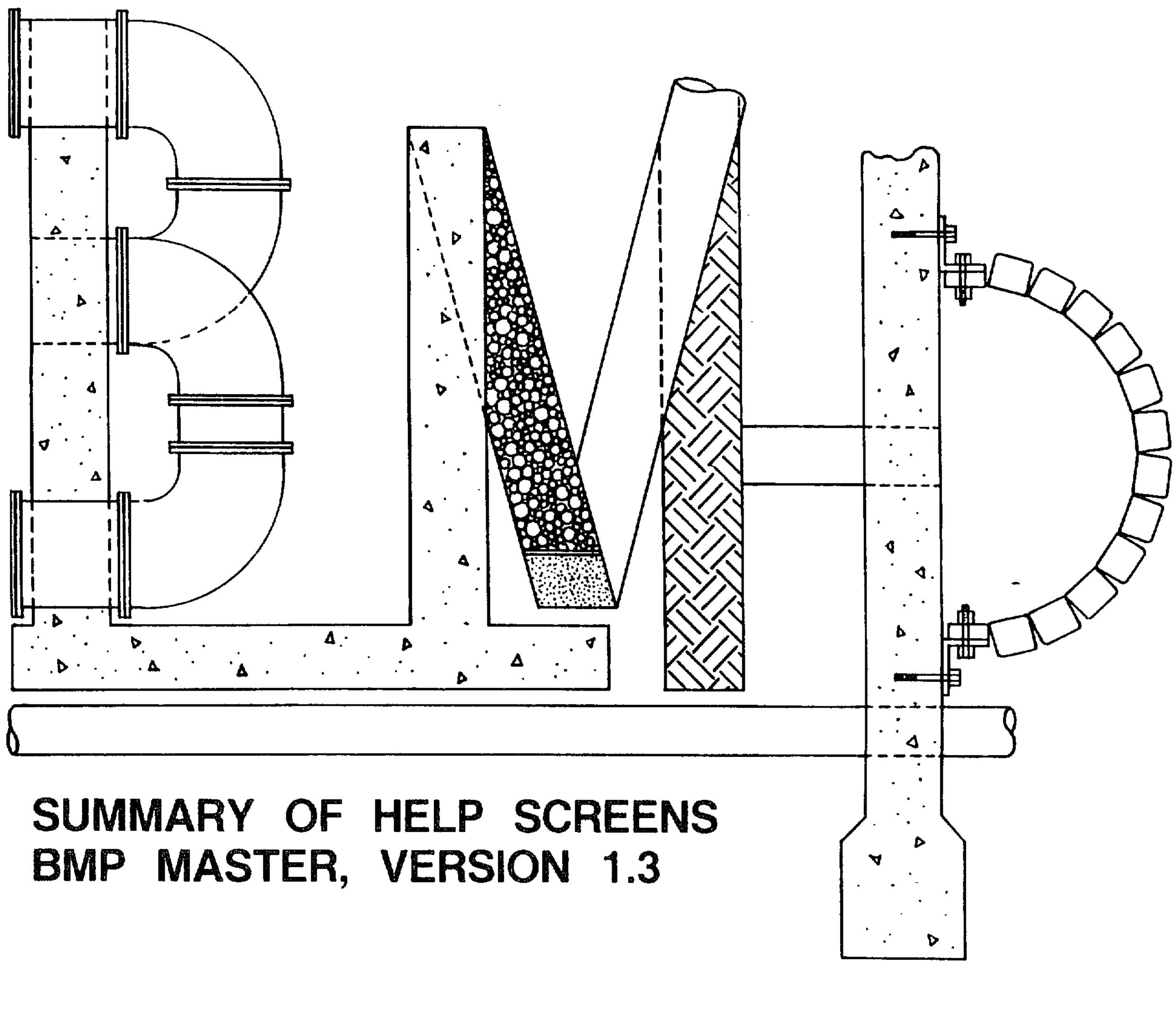
VIRGINIA LAND USE DATA — *FIGURE 3*

River Basin	total area (sq.mi.)	% URB	URB area (sq.mi.)	% FOR	FOR area (sq.mi.)	% PAST	PAST area (sq.mi.)	% CST	CST area (sq.mi.)	% CVT	CVT area (sq.mi.)
Potomac	14670	7	1027	56	8215	26	3814	7	1027	4	587
Rappahannock	2630	1	26	64	1684	20	526	8	210	7	184
York	2980	0.2	6	70	2090	13	388	10.1	302	6.7	200
James	10495	3	315	73	7661	14	1469	6	630	4	420
Eastern Shore	1000	1.5	15	50	500	805	85	9	90	31	310
Total (w/urban)	31781	5	1389	63	20150	20	6286	7	2259	5	1701
Total (w/o urban)	30398	n/a	n/a	66	20150	21	6286	7	2259	6	1701

URB = urban land uses
FOR = forest cover
PAST = pasture land
CST = conservation till acreage
CVT = conventional till acreage

Source: Commonwealth of Virginia, Council on the Environment, *Virginia's Chesapeake Bay Initiatives: First Annual Report* (Richmond, Va.: Council on the Environment, 1985).

BEST MANAGEMENT PRACTICES DESIGN GUIDANCE MANUAL
FOR HAMPTON ROADS



SUMMARY OF HELP SCREENS
BMP MASTER, VERSION 1.3

HAMPTON ROADS PLANNING DISTRICT COMMISSION

DECEMBER 1991

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SUMMARY OF HELP SCREENS

BMP MASTER, VERSION 1.3

December 10 , 1991

Prepared For:

Hampton Roads Planning District
Chesapeake, Virginia

Prepared by:

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```
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                         General Info
 ---------------------------------------------------
 Information regarding any questions, comments, or
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                    Smith Demer Normann
           Six Manhattan Square, Suite 102
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*E

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*P
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*E

*B 10,Site Criteria
          BMP Selection from Site Criteria
 ---------------------------------------------------
 This function will allow a general selection of
 feasible BMPs based on input site criteria.  The
```

selection process is for site planning purposes to determine which BMPs could be designed in the BMP Design module. The site criteria used include site area, ground water table, land slope, soil type, and proximity to drinking water wells. A report can be produced giving BMP site restrictions.

SEE ALSO:
^General Info^ ^BMP Design^ ^Removal Efficiency^
*E

*B 11,Removal Efficiency
BMP Selection from Removal Efficiency

This function will allow a general selection of feasible BMPs based on removal efficiency for a calculated pollutant loading increase. The selection process is used with the BMP Selection from Site Criteria to determine which BMPs could be designed in the BMP Design module. A report can be generated giving valid BMPs and the maximum removal efficiency ranges.

SEE ALSO:
^General Info^ ^BMP Design^ ^Site Criteria^
*E

*B 12,BMP Design
BMP Design

This function will allow the design of a BMP that is selected from a menu of feasible choices. The valid BMPs are made selectable by executing the BMP Selection from Site Criteria and also Removal Efficiency first. However, you can still choose to design a BMP that is not technically feasible from the list.

SEE ALSO:
^General Info^ ^Site Criteria^ ^Removal Efficiency^
*E

*B 13,Buffer Equivalency
Buffer Equivalency

In the Chesapeake Bay Preservation Act (CBPA), a 100 foot buffer is deemed to achieve 40% reduction of nutrients. This preliminary procedure determines the area needed for a BMP to provide reduction of pollutants from a new development site. The BMP thus determined can be installed in the buffer area or elsewhere on the site. The extent of the reduction of buffer area is the individual jurisdiction's decision.

< Page Down >
*P
For example, the buffer area might be reduced from 100 feet to 80 feet, with a BMP.

```
 SEE ALSO:
   ^General Info^  ^Efficiency of BMPs^
*E

*B 15,Edit Keys
                      Edit Keys
 ---------------------------------------------------------
 Cursor Keys- Move cursor direction of cursor key
 Ctrl-Left  - Move cursor word left
 Ctrl-Right - Move cursor word right
 Home       - Move cursor to beginning of field
 End        - Move cursor to end of field
 Ctrl-Home  - Move cursor to first field
 Ctrl-End   - Move cursor to last field
 Tab        - Next field   Shift-Tab   - prev Field
 Enter      - Next field   Ctrl-Enter  - Done

                        <PgDn>
*P
                   More Edit Keys
 ---------------------------------------------------------
 Insert     - Insert toggle
 Delete     - Delete character at cursor
 Backspace  - Delete character left

 Ctrl-Bckspc- Delete word left
 Ctrl-T     - Delete word right

 Ctrl-U     - Delete to end of field
 Ctrl-R     - Restore field with previous data
 Ctrl-Y     - Delete to end of form or last field
*E

*B 20,Area Served
                    Area Served
 ---------------------------------------------------------
 The area to be served by the BMP in Acres.  The
 area cannot be zero.




 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 21,Ground Water
                 Ground Water Table
 ---------------------------------------------------------
 The depth in feet of the seasonal high ground water
 table for the site.  The value cannot be zero.




 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E
```

```
*B 22,Slope
                     General Land Slope
 --------------------------------------------------
 The general slope of the land for the site, given
 in percent.  Example: 3.0%



 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 23,Well Proximity
                     Proximity to Wells
 --------------------------------------------------
 This is a Y or N question to determine if there are
 drinking water wells within 100' downslope of the
 BMP.



 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 24,Project Name
                        Project Name
 --------------------------------------------------
 This is your project title or name.  This can be
 used for a variety of purposes.  Some possible uses
 might be: job number, site name, alternative name.



 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 25,Soil Type
                         Soil Type
 --------------------------------------------------
 This is the soil type for the site.  The soil is
 selected from a pop-up menu by using the arrow keys
 or the first letter of the soil category name, and
 the ENTER key.  The soil types carry with them
 default infiltration values that are used later in
 the design process.


 SEE ALSO:
   ^General Info^  ^Soil Table^
*E

*B 26,Soil Table
 Soil Category Name   Inf Rate(In/Hr) Wtr Cap(In/In)
 ------------------   --------------- --------------
```

```
 Sand                          8.27            0.35
 Loamy Sand                    2.41            0.31
 Sandy Loam                    1.02            0.25
 Loam                          0.52            0.19
 Silt Loam                     0.27            0.17
 Sandy Clay Loam               0.17            0.14
 Clay Loam                     0.09            0.14
 Silty Clay Loam               0.06            0.11
 Sandy Clay                    0.05            0.09
 Silty Clay                    0.04            0.09
 Clay                          0.02            0.08
*E

*B 30,Site Area
                        Site Area
 ---------------------------------------------------
 This value can be either the physical area of the
 site in acres or the area of the site served by one
 particular BMP.  For example, if 50% of a 20 acre
 site drains to a BMP, you could input either:

            20 acres,  50% served
        or  10 acres, 100% served.

 IMPORTANT NOTE: If the area served is less than
 100%, you MUST compute the impervious cover for
                      < Page Down >
*P
                    Site Area (Con't.)
 ---------------------------------------------------
 just that portion of the site being served by the
 BMP.  In order to determine the overall removal
 requirement of the ENTIRE site, the loads from all
 the separate watersheds must be added together.




 SEE ALSO:
   ^General Info^ ^Edit Keys^ ^ImpCvrPost^ ^PctAreaServed^
*E

*B 31,ImpCvrWtrshed
                  Impervious Cover Pre
 ---------------------------------------------------
 If the site is a new development, then this value
 represents the % of impervious cover for the
 watershed that the site is a part of.  Defaults:
              16% -- Tidewater Area
              53% -- City of Norfolk
 If the site is being redeveloped, then this value
 represents the pre-redevelopment imperviousness of
 the site.

 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 32,ImpCvrPost
                  Impervious Cover Post
 ---------------------------------------------------
 This value is the percent impervious cover for the
 area listed in Site Area based on post-development
 conditions due to pavement, concrete, structures,
 etc.  REMEMBER, when the percent area served is
 less than 100, this percent impervious cover must
```

```
represent the impervious cover in the sub-watershed
actually being served by a selected BMP.  The value
in most cases will be greater than the Impervious
Cover Pre.
SEE ALSO:
  ^General Info^ ^Edit Keys^ ^Site Area^ ^PctAreaServed^
*E

*B 33,PctAreaServed
                Percent Area Served
---------------------------------------------------
This value can be either the physical area of the
site in acres or the area of the site served by one
particular BMP.  For example, if 50% of a 20 acre
site drains to a BMP, you could input either:

           20 acres,  50% served
       or  10 acres, 100% served.

IMPORTANT NOTE: If the area served is less than
100%, you MUST compute the impervious cover for
                 < Page Down >
*P
            Percent Area Served (Con't.)
---------------------------------------------------
just that portion of the site being served by the
BMP.  In order to determine the overall removal
requirement of the ENTIRE site, the loads from all
the separate watersheds must be added together.




SEE ALSO:
  ^General Info^ ^Edit Keys^ ^ImpCvrPost^ ^Site Area^
*E

*B 34,Redevelopment
                   Redevelopment
---------------------------------------------------
This is a Y or N question asking if the site is a
redevelopment.  Answering N implies the site is a
new development.




SEE ALSO:
  ^General Info^  ^Edit Keys^
*E

*B 35,Rainfall
              Average Annual Rainfall
---------------------------------------------------
This is the annual rainfall depth in inches.  It is
45 inches for the Hampton Roads area.




SEE ALSO:
```

```
  ^General Info^  ^Edit Keys^
*E

*B 40,OutFileName
                     Output File Name
 ---------------------------------------------------
 This is the pathname or device name specified for
 output used by reports.  Typical uses are:

   PRN               -- The standard printer.
   EXAMPLE.RPT       -- An example file for reporting.
   C:\BMP\TEST.OUT   -- A full pathname to a file.



 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 100,Biofiltration
                       Biofiltration
 ---------------------------------------------------
 Biofiltration as a BMP is similar to swale and
 filter strip BMPs.  Surface runoff can be treated
 by biofiltration to remove urban pollutants.  The
 runoff receives treatment through interaction with
 vegetation and the soil surface.  For a swale, the
 design depth of flow should be at least two (2)
 inches less than the winter vegetation height.
 Emergent wetlands plants can also be used to
 provide water quality benefits.

 SEE ALSO: ^General Info^  ^Efficiency of BMPs^
*E

*B 101,Dry Well
                         Dry Well
 ---------------------------------------------------
 The dry well is a variation of the infiltration
 trench and is designed exclusively for runoff from
 rooftops.  Roof leaders are extended to a stone
 aggregate filled trench located a minimum of 10
 feet from the building foundation.



 SEE ALSO: ^General Info^
   ^Infiltration Trench^  ^Efficiency of BMPs^
                     < Page Down >
*P
                       General Note
 ---------------------------------------------------
 The use of infiltration BMPs like Infiltration
 Trench, Dry Well, Porous Pavement, Infiltration
 Basin, Underground Storage, and Grid / Modular
 Pavement on fill material is not recommended.  Fill
 areas are susceptible to slope failure due to fill
 material becoming saturated when infiltration
 practices are used.



*E

*B 102,Infiltration Trench
                    Infiltration Trench
```

An infiltration trench is typically 3-8 feet deep and filled with stone aggregate to create an underground reservoir. Runoff can either drain from the reservoir into the underlying soil (exfiltration) or be collected by underdrains and directed to an outflow.

SEE ALSO:
^General Info^ ^Infiltration Basin^
^Dry Well^ ^Efficiency of BMPs^
< Page Down >

*P

General Note

Typically, infiltration trenches can only accommodate limited quantities of runoff and are used for sites of less than 10 acres in size.
The use of infiltration BMPs like Infiltration Trench, Dry Well, Porous Pavement, Infiltration Basin, Underground Storage, and Grid / Modular Pavement on fill material is not recommended. Fill areas are susceptible to slope failure due to fill material becoming saturated when infiltration practices are used.

*E

*B 103,Infiltration Basin

Infiltration Basin

Whereas infiltration trenches serve small sites, an infiltration basin can serve drainage areas up to 50 acres. They are designed to promote exfiltration through the underlying material. They should be vegetated and often include devices which prevent course sediment from entering the basin as well as emergency spillways for extreme storm events.

SEE ALSO: ^General Info^
^Infiltration Trench^ ^Efficiency of BMPs^
< Page Down >

*P

General Note

The use of infiltration BMPs like Infiltration Trench, Dry Well, Porous Pavement, Infiltration Basin, Underground Storage, and Grid / Modular Pavement on fill material is not recommended. Fill areas are susceptible to slope failure due to fill material becoming saturated when infiltration practices are used.

*E

*B 104,Grassed Swale

Grass Swale(w/Chk Dam)

Grassed swales are typically used in low density areas as an alternative to curb and gutter drainage systems. The pollutants are filtered out by the grass and subsoil. Check dams may be used to temporarily pond runoff, allowing infiltration over a period of time. They cannot, however, accommodate major runoff events and usually lead to other downstream BMPs.

SEE ALSO: ^General Info^
^Filter Strip^ ^Efficiency of BMPs^
*E

*B 105,Filter Strip
Filter Strip

Also known as buffer zones, filter strips are similar to grassed swales except that they are wider. They should be at least 20' wide and not used on slopes greater than 15%. The filter strips are usually forested and accept evenly distributed sheet flow. They can not accept channelized flow and function effectively. There are secondary benefits including aesthetics, wildlife habitat, and noise screening.
SEE ALSO: ^General Info^
^Grassed Swale^ ^Efficiency of BMPs^
*E

*B 106,Porous Pavement
Porous Pavement

Porous pavement detains and minimizes the effects of runoff containing traffic generated pollutants. It provides for removal by infiltration and bacterial action. It has a number of shortcomings which generally confine it to low volume traffic areas such as parking lots. It consists of a graded aggregate cemented by asphalt cement, with numerous voids providing a high permeability.
SEE ALSO: ^General Info^
^Grid/Modular Pavement^ ^Efficiency of BMPs^
< Page Down >
*P
General Note

The use of infiltration BMPs like Infiltration Trench, Dry Well, Porous Pavement, Infiltration Basin, Underground Storage, and Grid / Modular Pavement on fill material is not recommended. Fill areas are susceptible to slope failure due to fill material becoming saturated when infiltration practices are used.

*E

*B 107,Underground Storage
Underground Storage Trench

Underground storage trench is designed to remove sediments and hydrocarbons from parking lots and commercial sites where there is not enough space for infiltration systems. Underground storage trench, as a BMP should ONLY be installed when other BMPs are not feasible. It functions like an infiltration trench but can accept concentrated runoff. Unlike a surface trench, underground storage trench can be installed under the pavement of a parking lot. It is important and recommended
< Page Down >

*P
Underground Storage Trench<Con't.> 2

---

to pretreat the concentrated runoff before it enters the underground storage trench. The pretreatment can be accomplished by installing a water quality inlet upstream of the underground storage trench. If installed under the pavement of a parking lot, the pavement should be properly designed for the appropriate loadings.

While this BMP is not visible and can be aesthetically pleasing, maintenance and replacement costs can be prohibitive and costly.

< Page Down >

*P
Underground Storage Trench<Con't.> 3

Proper engineering judgement should be exercised in selection, design, and siting of this BMP.

SEE ALSO: ^General Info^
^Infiltration Trench^ ^Efficiency of BMPs^

< Page Down >

*P
General Note

The use of infiltration BMPs like Infiltration Trench, Dry Well, Porous Pavement, Infiltration Basin, Underground Storage, and Grid / Modular Pavement on fill material is not recommended. Fill areas are susceptible to slope failure due to fill material becoming saturated when infiltration practices are used.

*E

*B 108,Grid/Modular Pavement
Grid/Modular Pavement

Using the same concept as porous pavement, this type of pervious pavement consists of a grid made of concrete, clay bricks, or granite sets. The void areas of the grid are filled with a pervious material such as sod, gravel, or sand.

SEE ALSO: ^General Info^
^Porous Pavement^ ^Efficiency of BMPs^

< Page Down >

*P
General Note

The use of infiltration BMPs like Infiltration Trench, Dry Well, Porous Pavement, Infiltration Basin, Underground Storage, and Grid / Modular Pavement on fill material is not recommended. Fill areas are susceptible to slope failure due to fill material becoming saturated when infiltration practices are used.

*E

*B 109,Grit-Oil Separator

Grit-Oil Separator

Used to meet some of the water quality requirements where oil and grit deposits are likely such as in parking lot areas and commercial sites. A typical grit-oil separator consists of three chambers. The first two chambers maintain a permanent pool of water. The third chamber connects to the storm drain system or other infiltration BMP.

SEE ALSO:
^General Info^ ^Efficiency of BMPs^
*E

*B 110,Water Quality Inlet

Water Quality Inlet

Water quality inlets are typically used to serve parking lots one acre or less in size. A typical water quality inlet consists of one or two chambers. The water quality inlet is a smaller version of a grit-oil separator and functions in a similar fashion.

SEE ALSO:
^General Info^ ^Efficiency of BMPs^
*E

*B 111,Detention Pond

Detention Pond

Not available in this release, part of Phase II.

SEE ALSO:
^General Info^ ^Efficiency of BMPs^
*E

*B 112,Retention Pond

Retention Pond

Not available in this release, part of Phase II.

```
 SEE ALSO:
   ^General Info^  ^Efficiency of BMPs^
*E

*B 113,Extended Ret/Det Pond
                Extended Ret/Det Pond
 ---------------------------------------------------
 Not available in this release, part of Phase II.




 SEE ALSO:
   ^General Info^  ^Efficiency of BMPs^
*E

*B 114,DP/RP (Wetland Bottom)
               DP/RP (Wetland Bottom)
 ---------------------------------------------------
 Not available in this release, part of Phase II.




 SEE ALSO:
   ^General Info^  ^Efficiency of BMPs^
*E

*B 200,Infiltration Rate
                  Infiltration Rate
 ---------------------------------------------------
 The infiltration rate can be pre-selected by first
 executing the BMP Selection from Site Criteria.
 Choose the appropriate Site soil type.  A different
 value can also be input, based on information from
 County Soil Survey or site soil survey.



 SEE ALSO:
   ^General Info^  ^Soil Table^  ^Edit Keys^
*E

*B 201,Max. Storage time
       Maximum Allowable Ponding/Storage Time
 ---------------------------------------------------
 The maximum allowable ponding/storage time is the
 time for which a BMP is designed to completely
 drain.  72 hours is the maximum (and default)
 value, but other values can also be input.



 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E
```

*B 202,Void Ratio

Void Ratio

The void ratio is the ratio of voids to the volume of the solids. 0.4 is the common void ratio and is the default value. The value cannot be zero nor can it be greater than 1.

SEE ALSO:
^General Info^ ^Edit Keys^

*E

*B 203,Min. Vert. Dist.

Minimum Distance From BMP Bottom to Ground Water

The minimum distance from the BMP bottom to the ground water table is defaulted at a recommended minimum of 2.0 feet. However, the user can design with any value greater than 2.0. The Virginia Stormwater Management Regulations require that the invert of the infiltration BMPs should be four (4) feet above the seasonal high groundwater table.

SEE ALSO:
^General Info^ ^Edit Keys^

*E

*B 204,Runoff Depth Increase

Increase in Runoff Depth

The increase in runoff depth is the change in runoff (increase) that will result from site development/improvement. The designer will have to compute this using currently accepted hydrologic methods and convert the runoff to depth.

For Dry Well, increase in runoff depth is from the Rooftop area.

SEE ALSO:
^General Info^ ^Edit Keys^

*E

*B 205,Contributing Drainage Area

Contributing Drainage Area

This is the total site area (in square feet) that contributes runoff to the BMP. This value must be greater than zero.

For Dry Well, this area is the Rooftop area.

SEE ALSO:
^General Info^ ^Edit Keys^

*E

*B 206,BMP Depth

BMP Depth

This is the depth to which the BMP is to be designed. The maximum value is determined in the feasibility window encountered immediately prior to this design window and is automatically defaulted ahead.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 207,Rainfall Amount
Amount of Rainfall

This is the depth of rainfall associated with the design storm. Some examples are:

2-Year storm=3.0 inches
10-Year storm=5.0 inches

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 208,BMP Filling Time
BMP Filling Time

This is the time in hours for the BMP to fill. The default value for Dry Well is one(1) hour. For other infiltration BMPs, the default value is two(2) hours. The designer is responsible for determining this time based on accepted hydrologic time of concentration methods.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 209,Basin Side Slopes
Basin Side Slopes

The desirable side slopes (horizontal to vertical ratio) for the infiltration basin. This represents the side slopes for the entire basin. Consideration of variable side slopes is not possible in this program.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 210,Basin Top Width
Basin Top Width

The desirable top width of the infiltration basin.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 211,Depth of Subbase Aggregate
Depth of Subbase Aggregate

The design depth of stone aggregate reservior.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 212,Runoff Depth from Well Area
Runoff Depth from Area Over Dry Well

The depth of stormwater runoff contributed only by that area over the dry well.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 213,Depth of Soil Over Well
Depth of Soil Overlying Dry Well

The depth of the cover material over the BMP. In the case of the Dry Well this represents the soil cover and typically is one foot. For the Underground Storage BMP, this cover could possibly be pavement, soil, or concrete.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 214,Water Capacity of Soil
Water Capacity of Soil Overlying Dry Well

The effective water capacity of a soil is the fraction of the void spaces available for water storage, measured in terms of inches per inch.

```
 SEE ALSO:
   ^General Info^  ^Soil Table^  ^Edit Keys^
*E

*B 215,Discharge Rate
       Discharge Rate (Cubic Feet per Second)
 ---------------------------------------------------
 The discharge for which the Biofiltration is being
 designed.  The discharge could be the discharge
 associated with one-inch(1") rainfall or two(2)
 year rainfall.




 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 216,Manning's 'n' Value
     Manning's Coefficient of Vegetation Cover
 ---------------------------------------------------
 The roughness coefficient used with the Manning's
 equation.  Typical values for vegetation are:

   Dense grass up to 6" tall            -- 0.07
   Dense grass 6" - 12" tall            -- 0.10
   Dense grass    > 12" tall            -- 0.20
   Vegetation with coarser stems
    (wetland plants, woody plants, etc.) -- 0.07

 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 217,Depth of Flow
          Depth of Flow in Parabolic Swale
 ---------------------------------------------------
 The depth of the flow for which the Biofiltration
 swale is being designed.  It is recommended to use
 the design depth of flow at least two(2) inches
 less than the winter vegetation height.




 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 218,Longitudinal Slope
                Longitudinal Slope
 ---------------------------------------------------
 The ground slope along the water flow path in the
 Biofiltration swale or Grass swale with check dams.




 SEE ALSO:
```

^General Info^ ^Edit Keys^
*E

*B 219,Length of Swale
Length of Swale

The longitudinal length of the Biofiltration swale. Typically the length is 200', but site constraints may require a shorter length to be used. In such cases, use a length less than 200', for example 150'.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 220,Bottom Width Check Dam
Bottom Width of Check Dam

The bottom width in feet of check dam.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 221,Side Slope
Side Slope

The ratio of the swale side slope. A typical value might be 3 or 4 to signify 3:1 or 4:1 respectively.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 222,Total Hydraulic Length
Total Hydraulic Length of Swale

The total hydraulic length of swale in feet can be input by the user or calculated internally. Simply input the known value if known or input a 0 in the field and process the screen with Ctrl-Enter. Another window will pop up allowing the input of parameters that will be used to calculate the hydraulic length to be used.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 223,Max. Ponding time

Maximum Allowable Ponding Time

The maximum allowable ponding time is the time for which a Grassed Swale is designed to completely drain. 24 hours is the maximum (and default) value, but other values can also be input.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 300,Contributing Impervious Area
Contributing Impervious Drainage Area

This is the impervious area in acres which is contributing runoff to the grit-oil separator. Roof surface area can be neglected.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 301,Length of Chamber
Length of Chamber

This is the length of the first chamber in a grit-oil separator or a water quality inlet. The minimum recommended length is six (6) feet. A higher value can be input.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 302,Width of Chamber
Width of Chamber

This is the width of the first chamber in a grit-oil separator or a water quality inlet. The minimum recommended width is 2'-6". A higher value can be input.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 303,Contributing Flow from Impervious Area
Contributing Flow from Impervious Area

This value is the runoff generated from the

contributing impervious area. The runoff in cubic feet per second (cfs) is the "first flush" flow for the curb inlet opening or 10-year storm entering the grit-oil separator from the storm drain system.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 304,Diameter of Drawdown Pipe
Diameter of Drawdown Pipe

This value is the diameter of the drawdown pipe in inches. This pipe in a grit-oil separator connects the second and third chambers. In a water quality inlet this is the outflow from the main chamber. The minimum diameter recommended is six (6) inches. A larger size drawdown pipe can be used. The pipes can be cast iron or aluminized corrugated metal pipe.

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 305,Number of Drawdown Pipes
Number of Drawdown Pipes

This value is the number of drawdown pipes connecting the second and third chambers in a grit-oil separator or from the main chamber of a water quality inlet. The minimum recommended number of drawdown pipes is two (2).

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 306,Free-Board Value
Free-Board Value

The minimum recommended free-board value is 1'-6".

SEE ALSO:
^General Info^ ^Edit Keys^
*E

*B 320,Efficiency of BMPs
Efficiency of BMPs(%)

(40-80) BioFiltration
(50-70) Dry Well
(50-70) Infiltration Trench
(50-70) Infiltration Basin

```
     (10-20) Grass Swales(w/ chk dams)
     (20-50) Filter Strips
     (50-70) Porous Pavement
     (50-70) Underground Storage
     (50-70) Grid/Modular Pavement
     (10-25) Grit-Oil Separator
     (10-25) Water Quality Inlet    < Page Down >
*P
           Efficiency of BMPs(%) <Con't.>
 ---------------------------------------------------
     (20-50) Detention Ponds
     (35-65) Retention Ponds
     (25-60) Extended Det/Ret Ponds
     (40-75) Det/Ret w/ Wtlnd Btms




 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 323,Design For First Flush
               Design For First Flush
 ---------------------------------------------------
 The "first flush" is the runoff associated with the
 most frequent storms.  "First flush" designs can be
 developed for :
   (1) The runoff produced by a one-inch (1") storm
       over the contributing site area.
   (2) 0.5 inch of runoff per impervious acre in the
       contributing site area (first flush).
   (3) The runoff per impervious acre produced by a
       one-inch (1") storm.

                     < Page Down >
*P
          Design For First Flush <Con't.>
 ---------------------------------------------------
   (4) 0.5 inch of runoff in the contributing site
       area (Virginia Stormwater Management
       Regulations).

 A large percentage of urban pollutants being
 discharged into receiving waters are associated
 with most frequent storms (normal rainfall).


 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E

*B 999,How To Quit
                  Quit BMP Master
 ---------------------------------------------------
 This will take you back to the DOS environment. You
 will first be asked if you are sure that you want
 to quit.

   Possible Responses:

     'N' Key     :  Returns to main menu.
     ESCAPE Key  :  Returns to main menu.
     ENTER Key   :  Returns to main menu.
     'Y' Key     :  Quits program.
*E
```

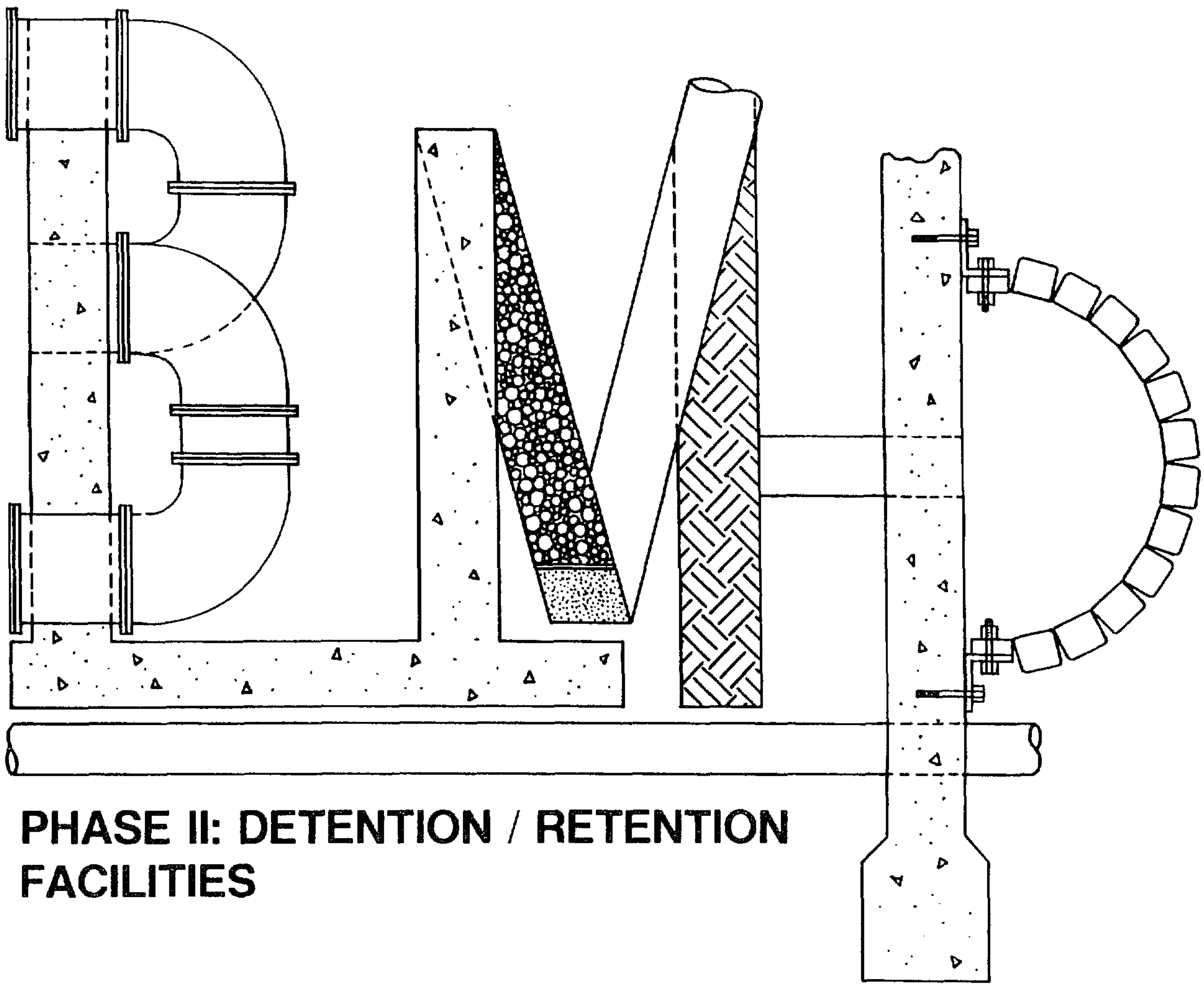
```
*B 1000,Report Help
                Report File Pathname
 ---------------------------------------------------
 This is a file name or device name where you want
 your reports to go.

   Common Uses are:

     PRN           :   The standard printer.
     BMP.RPT       :   A sample report file name.
     C:\TEST.FILE  :   A sample full pathname.

 SEE ALSO:
   ^General Info^  ^Edit Keys^
*E
```

BEST MANAGEMENT PRACTICES DESIGN GUIDANCE MANUAL
FOR HAMPTON ROADS



PHASE II: DETENTION / RETENTION FACILITIES

HAMPTON ROADS PLANNING DISTRICT COMMISSION

DECEMBER 1991

BEST MANAGEMENT PRACTICES

DESIGN GUIDANCE MANUAL

FOR

HAMPTON ROADS VIRGINIA

PHASE II: DETENTION/RETENTION FACILITIES

This report was produced, in part, through financial support from the Virginia Council on the Environment pursuant to Coastal Resources Management Program Grant No. NA90AA-H-CZ796 from the National Oceanic and Atmospheric Administration and from the Chesapeake Bay Local Assistance Department pursuant to Contract No. 91-42 of August 1990 and Unnumbered Contract of June 20, 1990.

Preparation of this report was included in the HRPDC Work Program for FY 1990-91, approved by the Commission at its Executive Committee Meeting of March 21, 1990 and in the HRPDC Work Program for FY 1991-92, approved by the Commission at its Executive Committee Meeting of March 20, 1991.

Prepared by URS Consultants, Inc.
in cooperation with the Staff of the
Hampton Roads Planning District Commission

December 1991

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February 4, 1992

Mr. John Carlock
Hampton Roads Planning District Commission
The Regional Building
723 Woodlake Drive
Chesapeake, Virginia 23320

RE: Detention - Retention
BMP Design Guidance Manual

Dear Mr. Carlock:

The attached document is submitted in accordance with our contract of November 4, 1991. The document was produced in accordance with our discussion and follows the format established by the HRPDC. Because one of the considerations was to have sections that could be removed and stand alone, there is duplication of some parts of Sections 3 and 4.

As we discussed during the preparation of the document, the focus is on guidance rather than design examples. It is expected that the user will have the knowledge and experience to prepare the design. Further, the variables in design of retention and detention facilities and the need to have design flexibility and innovation limit the value of details or step by step procedures. The state of the art is advancing rapidly and computer software is becoming more sophisticated, which also strengthened the decision to produce a guidance document.

This document is in a final draft form for the review of your staff and the committee. Consequently, the table of contents and the lists of tables and figures have not been finalized because we are certain that changes in page numbers will be made. Also, we are still in the process of receiving some data which we believe would be valuable input, and we will include that data in the final report.

We have truly enjoyed the opportunity to work with you on this project and trust that the fruits of our labor will be beneficial to all of the users.

Very truly yours,

URS CONSULTANTS, INC.

Lamont W. Curtis, P.E.
Vice President

Enclosures

LWC/kbr

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The Hampton Roads Planning District Commission, on behalf of its member local governments has produced a Stormwater Management Strategy for managing and financing programs. The strategies were developed to assist the communities in preparing Stormwater Management Programs consistent with the USEPA NPDES Stormwater Program, the Chesapeake Bay Preservation Act, the State Erosion and Sediment Control Program, and the State Stormwater Management Regulations. To further assist, HRPDC has prepared two documents for guiding the design of on-site and regional best management practice (BMPs) stormwater facilities. This document addresses retention and detention basins for use as both local and regional BMPs.

The purpose of this document is to provide general design guidelines. The focus is on selecting a design that will satisfy the Chesapeake Bay Preservation Act performance criteria, provide stormwater management consistent with the Stormwater Management Regulations, and be cost effective in construction and maintenance costs.

Prior to the 1960's, the focus in stormwater management was on drainage and flood control. Getting the water off the streets and out of the yards was the primary concern. Little attention was paid to the consequences downstream within the developing area or to impacts on developed neighborhoods. Flood control was a problem relegated to the Government and to agencies such as the Corps of Engineers, Bureau of Reclamation and Soil Conservation Service. In the 1960's, increasing urbanization was recognized as a serious contributor to local flooding problems. These problems included taxing the capacity of the local streams, causing increases in flood stages downstream, erosion of channel banks, and causing peak flow problems at wastewater plants through infiltration and inflow into the sanitary sewers. The courts, in general, held little hope for the suffering downstream property owner in the eastern part of the United States. Figure 1-1 pictures the problems associated with urbanization.

To solve these problems, many turned to the concept of detention and retention basins. They had been used successfully on a large scale at the Miami Conservancy District in Ohio, the Tennessee Valley Authority, and were promoted for rural areas by the Soil Conservation Service. In those areas which started to use retention and detention, the initial concept was to store water to reduce peak storm flows to some manageable rate of flow.

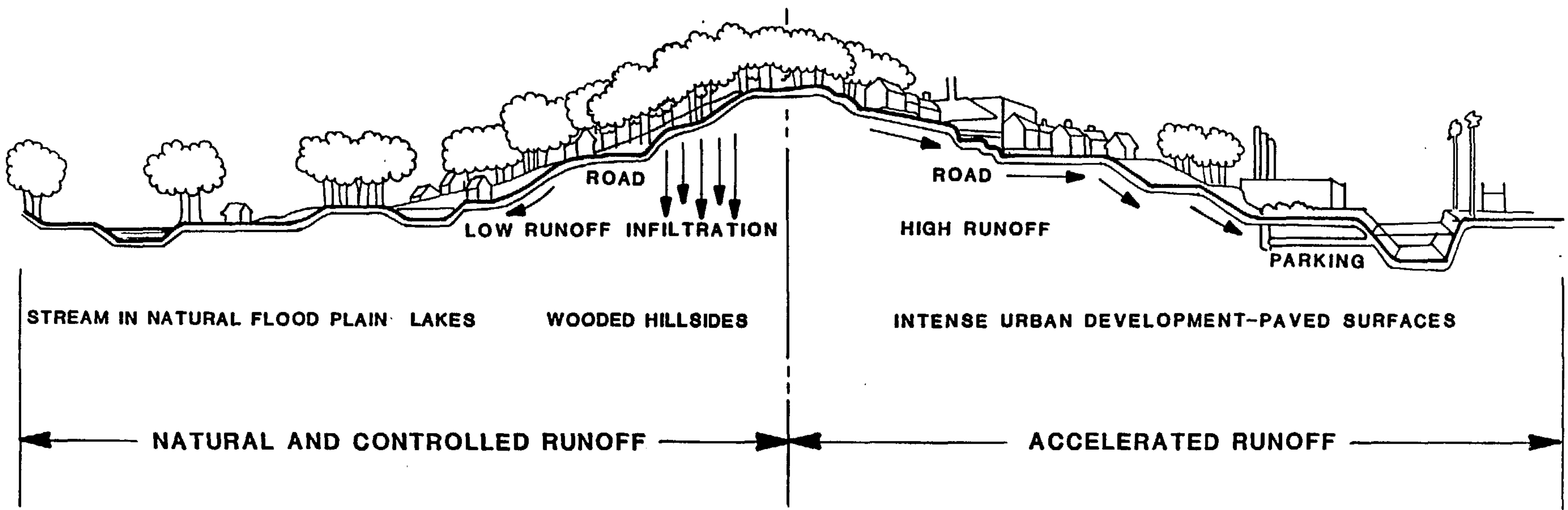

FIGURE 1-1

URBANIZATION ACCELERATES STORMWATER RUNOFF

(Source.- Ohio Stormwater Control Guidebook, Ohio Department of Natural Resources, 1980. p.11)

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Many political subdivisions and localities passed laws and wrote regulations requiring a reduction in flow rates to a pre-development level. A result of this was the proliferation of basins with no regard for the hydraulics of the watershed or the potential compounding effect of the reduced but elongated outflow hydrographs as they are routed downstream. Further, another more basic question of who was going to maintain these basins when the developer moves on to his next project was not always considered. These practices have shown that watershed planning is a critical element in locating regional facilities, and the ownership and maintenance of the basin needs to be addressed in the initial planning.

The passage of the Clean Water Act, Public Law 92-500, in 1972 by Congress showed the nation's concern with pollution and water quality issues. Several studies resulting from that Act such as the Nationwide Urban Runoff Program, 208 Water Quality programs, and the Urban Studies program pointed to non-point source runoff as a major contributor to the total water quality picture. Many investigations pointed to the benefits of detention storage and the settling that takes place as a positive attribute of detention and retention basins. More recently, the incorporation of wetland features have proved to be another added benefit of pollution reduction and habitat preservation. Further, landscape architects and urban planners have incorporated these basins into aesthetic and recreational features of developments.

As a result, the design of a retention or detention basin ought to consider the advantages of a multi-objective facility. Research is ongoing and experiences, both successes and failures, are increasing our knowledge and advancing the state-of-the-art of detention and retention basin design. Consequently, it is expected that this manual will be a dynamic document. Changes will be made as experience dictates.

1.2 GENERAL PLANNING CONSIDERATIONS

Regional detention or retention basins control stormwater runoff from large areas. When best management practices for reducing non-point source pollution are added, the facility becomes a Regional Best Management Practice (BMP) or Stormwater Management Facility (SWMF). In this document, detention or retention basins designed to service a large area or a regional area will be referred to as a Regional Stormwater Management Facility. The volume of water is typically greater than that which can be handled by on-site facilities. The watershed area is typically measured in acres and even square miles. There is no universal definition for a regional SWMF, but in general, a regional SWMF would control runoff peaks and enhance water quality from a watershed large enough such that a detailed engineering study would be necessary to evaluate the hydraulic impact on downstream areas. The design usually incorporates multiple objectives of flood control, water quality enhancement, aesthetics, recreation, wildlife habitat protection, habitat management, and groundwater recharge and protection. Multi-objective planning is a change from the traditional approach. Not only does it satisfy several objectives, but it enhances the neighborhood's and community's attractiveness and character. Figure 1.2 shows the concept of multi-objective stormwater management planning.

The location and design of regional SWMFs need to be the subject of a watershed management plan. Location within a watershed and release rates need to consider the routing of flows so as not to accumulate rates of flow above those which cause erosion and flooding. A Regional Stormwater Management facility should not only protect against problems associated with further development but should be designed whenever possible to reduce problems caused by prior development. CBLAD notes that regional facilities must protect against downstream flooding and erosion regardless of "normal" conditions. Regional SWMFs also require more land, so it is incumbent upon the planner to identify future sites which can be protected or purchased. The selection of the design features to incorporate in the BMP depends upon the need for water quality enhancement, peak discharge reduction requirements, and other criteria of the multiple objectives selected.

The operation and maintenance issues of several smaller retention-detention facilities versus a larger regional facility needs attention in the selection process. This particular criteria is very important because neither the Virginia Stormwater Management Regulations or the Chesapeake Bay Preservation Act speaks to the resolution of ownership and maintenance responsibility. In general, both regulations require a commitment that maintenance be addressed, but do not require that the property owner or locality take on the responsibility.

Section 3.8.8 of the Virginia Stormwater Management Regulations call for six planning steps as a minimum which are as follows:

1. "Consideration of the locality's comprehensive plan, zoning, government facility plans and similar planning tools."

2. "An analysis of the impacts of development on the watershed based on hydrologic and hydraulic modeling. At a minimum, the 2-year, 10-year, and 100-year storms shall be studied. Ultimate development of the watershed shall be assumed."

3. "Recommendations for locations, specified release rates, and required storage capacities of needed regional stormwater management facilities based on the modeling."

4. "Consideration of future expansion of regional stormwater management facilities based on the possibility that development might exceed the anticipated level."

5. "Requirements for necessary onsite stormwater management facilities

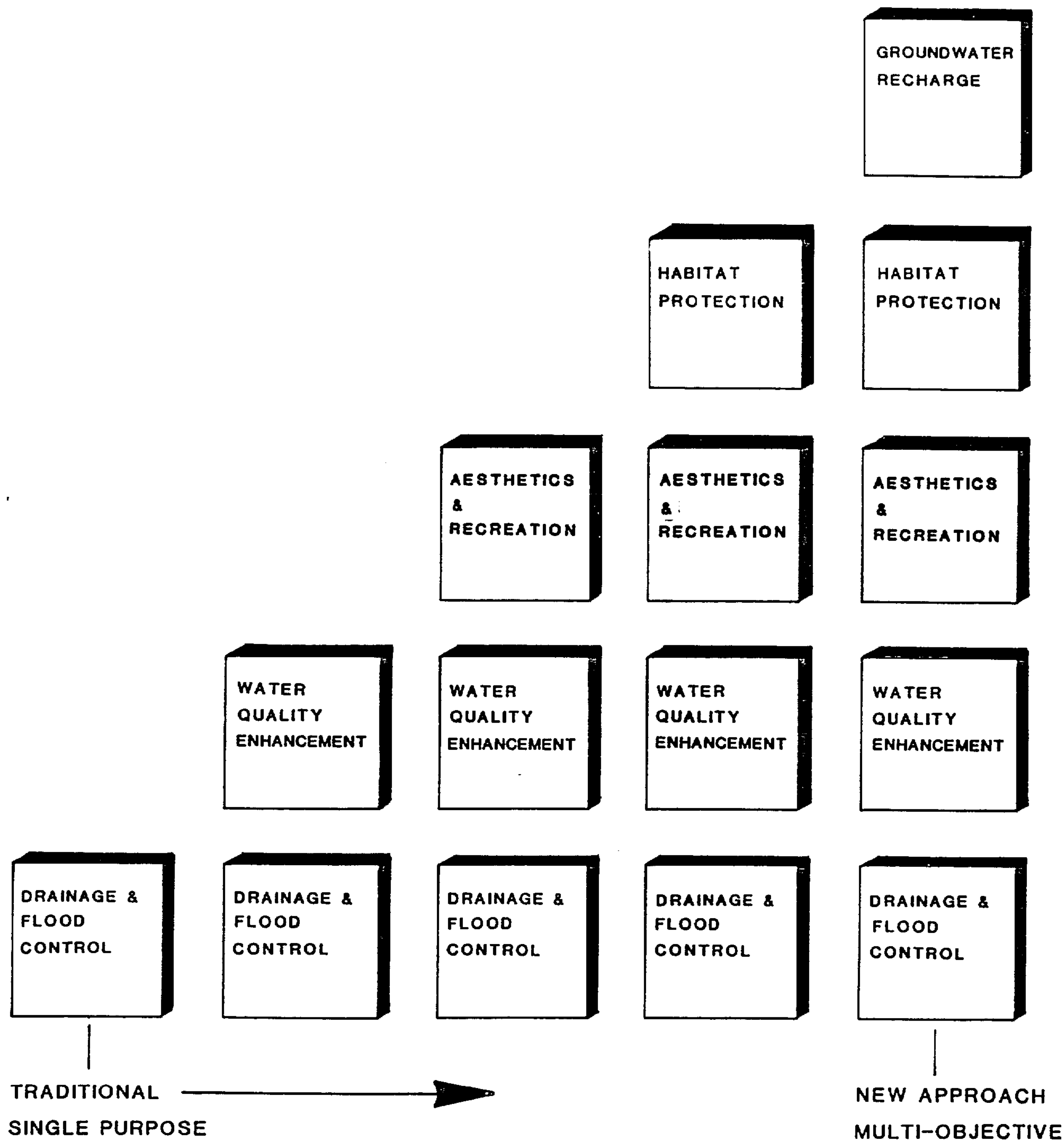

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FIGURE 1-2

MULTI-OBJECTIVE PLANNING

(Source- Clinton River Watershed Planning Guidelines)

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and release rates."

6. "An implementation schedule and financing requirements."

Within much of the Hampton Roads area, the flat topography limits the area that can be effectively drained to a regional SWMF. The high groundwater level and poor soils limit the use of infiltration type systems and also reduce the volume of storage available in basins. The high groundwater level will also impact the design of retrofitting basins in order to create a shallow pond for enhancing water quality improvement.

1.3 REGULATIONS REGARDING BMPs

Within the Hampton Roads area, stormwater is regulated by the Commonwealth primarily under the Stormwater Management Regulations (VR 215-02-00), the Chesapeake Bay Preservation Area Regulation (VR 173-02-01) and the Virginia Erosion and Sediment Control Regulations (VR 625-02-00). The Stormwater Management Regulations result from Article 1.1 of Chapter 6 of Title 10.1. The Chesapeake Bay Act is Chapter 21 of Title 10.1, and the Virginia Erosion and Sediment Control Law is found in Title 10.1 Chapter 5 Article 4 of the Code of Virginia. In addition to these regulations, the Dam Safety Act may become important if the impoundment exceeds 50 acre feet and the dam is over 25 feet high. The USEPA Stormwater NPDES regulations will have an impact. Other regulations may include state and federal wetlands permits, 401 Water Quality Certification and State Water Control Board Regulations. Pending regulations of the Coastal Zone Management Act Reauthorization - Coastal Non-point Program may have a future impact.

It has been recognized that these laws, regulations, and permits overlap, and the staffs of the state departments are working to consolidate the requirements to reduce any conflicts and to develop a checklist for submittals.

A brief summary is included to provide a general description of the key regulations.

1.3.1 STORMWATER MANAGEMENT ACT AND REGULATIONS

The purpose of the Virginia Stormwater Management Act, enacted in 1989, is to enable localities to inhibit the deterioration of existing water quality and maintain runoff at pre-development characteristics as nearly as practical.

The Stormwater Management Regulations are applicable to those localities that establish a local stormwater management program and every state agency that disturbs land and soil. There are several exemptions including single family homes separately built, and land development projects that disturb less than one acre of land.

The Regulations establish technical criteria for new development which local programs must meet as a minimum, encourages watershed planning, sets up administrative procedures, and establishes maintenance as an important feature that needs to be considered.

Specific Technical Criteria of importance to retention - detention facilities are:

Storage and Outlet Discharge - If retention or detention is used solely or in combination with other stormwater management practices, the end result is that the post development rates from the 2 year and 10 year storm do not exceed the pre-development rate, as nearly as practical.

Water Quality Volume - Water Quality Volume is defined as the first 0.5 inches of runoff per acre of the land development project, which is the area subject to manmade changes. In a detention basin, this volume needs to be detained and released over thirty hours from the time of peak storage. In a retention basin, the permanent pool needs to be three times the water quality volume as a minimum.

Design Storm - Using Soil Conservation Service (SCS) methodology, the 24-hour rainfall distribution recommended by SCS is used. When using other methods, the rainfall intensity curves for the appropriate return interval are to be used with the duration of the design rainfall intensity occurring over a period equal to the time of concentration. Other durations need to be checked for maximum volumes.

The Regulations also encourage watershed planning and require that permanent arrangements satisfactory to the approving agency be prepared for operation and maintenance.

1.3.2 CHESAPEAKE BAY PRESERVATION ACT AND REGULATIONS

The Chesapeake Bay Preservation Act was enacted in 1988 to protect water quality in the Chesapeake Bay. The Act and Regulations establish Land Use and Development Performance criteria to reduce the contribution of nonpoint source runoff being transported to the Chesapeake Bay by stormwater runoff. The objectives of the criteria (Section 4.1) are to:

a. "Prevent a net increase in nonpoint source pollution from new development."

b. "Achieve a 10% reduction in nonpoint source pollution from redevelopment."

c. "Achieve a 40% reduction in nonpoint source pollution from agricultural and silvicultural uses."

The Performance Criteria include several elements. Among them are the following:

- "Where the Best Management Practices utilized require regular or periodic maintenance in order to continue their functions, such maintenance shall be ensured by the local government through a maintenance agreement with the owner or developer or some mechanism that achieves an equivalent objective." (Section 4.2.3)

- "Stormwater management criteria which accomplish the goals and objectives of these regulations shall apply. For development, the post-development nonpoint source pollution runoff load shall not exceed the pre-development load based upon average land cover conditions. Redevelopment of any site not currently served by water quality best management practices shall achieve at least a 10% reduction of nonpoint source pollution in runoff compared to the existing runoff load from the site. Post-development runoff from any site to be redeveloped that is currently served by water quality best management practices shall not exceed the existing load of nonpoint source pollution in surface runoff." (Section 4.2.8)

- Performance is measured by use of the "Keystone Pollutant" concept. The concept simplifies the computations of loads and is generally accepted as being a practical and realistic indicator of total nonpoint source pollution loads. Total phosphorus has been selected as the keystone pollutant in Virginia.

- A manual has been prepared which includes substantial detail on the application of the regulations and on calculating the performance of BMPs to determine compliance with The Chesapeake Bay Preservation Act. The Guidance Calculation Procedure is included as Appendix B. The entire manual entitled Local Assistance Manual (November 1989) can be purchased from the Chesapeake Bay Local Assistance Department (CBLAD). It has been provided to all affected localities by CBLAD.

1.3.3 VIRGINIA EROSION AND SEDIMENT CONTROL LAW AND REGULATION

The Erosion and Sediment Control Law, enacted in 1973, is established to control soil erosion, sediment deposition, and non-agricultural runoff. Minimum standards have been established by the regulations that require techniques and methods to be employed to meet the criteria. The sections of the Regulations which pertain directly to retention-detention stormwater management facilities are:

- Sediment basins and other measures intended to trap sediment shall be constructed as a first step. (Section 1.5.4)

- Surface runoff from three or more acres, which runs across disturbed areas of 10,000 square feet or more, shall be controlled by a sediment basin.

- Downstream waterways shall be protected from damage due to increases in volume, velocity and peak flows from two- and ten-year frequency storm events.

- A plan for maintenance needs to be approved.

- A handbook providing guidance has been prepared and is available from the Division of Soil and Water Conservation, Department of Conservation and Recreation. The most recent edition is the second edition, dated 1980. This handbook is currently being revised.

1.3.4 VIRGINIA DAM SAFETY ACT AND REGULATIONS

This Act and Regulations provide for the safe design, construction, operation and maintenance of impounding structures to protect public safety. The Regulations establish specific design criteria. The Act and Regulations include all dams which are equal to or greater than 25 feet high as measured as a vertical distance from the natural bed of the stream at the downstream end to the top of the impounding structure and which create a maximum impoundment equal to or greater than fifty acre feet. The top of the impounding structure is defined as the lowest point of the non-overflow section of the impounding structure.

1.3.5 USEPA STORMWATER NPDES PERMIT

Regulating stormwater has been a controversial subject since the 1972 Clean Water Act. When the Act was reauthorized in 1987 by the Water Quality Act of 1987, provisions were included to govern stormwater discharge through a phased approach to establish permits for stormwater discharges. The final regulations were published on November 16, 1990, for the NPDES permit application requirement. The Regulations require that pollutants in stormwater discharges, for both existing systems and new systems associated with development, be reduced to the "maximum extent practicable."

The permit is applied for in two parts - PART I and PART II. In PART I, the section Source Identification is intended to identify possible sources of pollutants to the separate storm sewer system and to identify possible locations for treatment based controls. In this section any retention or detention basin needs to be identified so that it can be studied for retrofit for use as a BMP. Also public lands need to be identified for new structures.

Also in PART I is a section on Discharge Characterization for the purpose of identifying existing short- and long-term water quality problems in stormwater discharges. The PART I section on Management Plans requires identification of existing structural and non-structural programs to control pollution from stormwater. The description shall provide information on operation and maintenance as well.

PART II of the application is submitted after PART I and goes into more detail. It requires submission of a proposed management plan describing how the applicant proposes to improve the water quality of its stormwater runoff. Examples of these programs include:

Stormwater Regulations
Erosion and Sediment Control Regulations
Clean Water Connections to Sanitary Sewers
Floodplain Management
On-site Stormwater Control
Stormwater Management Plans
Non-Structural Controls
Public Education Programs

1.4 CURRENT PRACTICES BY JURISDICTIONS

TABLE 1-1

CURRENT PRACTICES BY JURISDICTIONS

JURISDICTION	STORMWATER MANAGEMENT	CBPA
Chesapeake		in place
Franklin	Follows State	not applicable
Hampton		in place
Isle of Wight County		in place
James City County		in place
Newport News		in place
Norfolk		in place
Poquoson		in place
Portsmouth		in place
Smithfield		in place
Southampton County		not applicable
Suffolk		in place
Virginia Beach	Follows State	in place
Williamsburg		in place
Windsor		in place (2/92)
York County		in place

1.5 DEFINITIONS

The Virginia Department of Conservation and Recreation, Division of Soil and Water Conservation has defined many elements of a stormwater management program. For the sake of consistency this manual will use many of the same definitions but will make modifications for clarity rather than substantial change.

The definitions are applicable in a general sense. Each locality may have different regulations, and it must be realized that the design of these facilities needs to be in accordance with that specific locality's Stormwater Management Program.

Regional BMP: a facility or practice designed to function as a best management practice (BMP) for an area ultimately encompassing more than one property owner. These facilities are designed to control water from a large contributing area, although only portions of the watershed may experience land development. These facilities are best planned as a result of a regional or watershed-based stormwater management plan.

Regional stormwater management plan or regional plan: means a document containing material describing how runoff from open space, existing development and future planned development areas within a watershed will be controlled by coordinated design and implementation of regional stormwater management facilities.

Stormwater management facility: means a device that controls stormwater runoff and changes the characteristics of that runoff including, but not limited to, the quantity and quality, the period of release or the velocity of flow.

Stormwater management plan or plan: means a document containing material for describing how existing runoff characteristics will be maintained by a land development project and will comply with the requirements of the local program or the State regulations.

Retention Basin: a stormwater management facility comprised of: a) a permanent pool of water which loses water primarily through infiltration and evaporation which may be increased in volume to enhance water quality; and b) additional capacity above the permanent pool for the storage of stormwater runoff. The facility discharges to the downstream conveyance system through an outlet structure designed to both release the runoff over a specified period of time and to maintain the permanent pool at a minimum level. These facilities are also called wet ponds or wet detention basins and can be used for both stormwater quantity and quality control.

Detention Basin: a stormwater management facility which temporarily stores runoff, with discharge to the downstream conveyance system through an outlet structure designed to completely empty the facility over a short time period, typically six hours or less. These facilities, also called dry ponds, are used primarily to control runoff quantity, however, some water quality enhancement occurs through sedimentation.

Extended release: the use of a modified outlet structure in either a detention or retention basin to extend the stormwater runoff storage time beyond that typically used for quantity control and achieve water quality enhancement through nonpoint source pollutant removal.

Wetlands Bottom: the establishment of a wetland or shallow marsh area in detention or retention basins to enhance the removal of soluble pollutants, enhance sediment trapping, reduce sediment resuspension, and conceal trash and debris.

Water Quality Storage: the storage in the permanent pool of a retention basin to meet the regulatory requirements.

Infiltration facility: means a stormwater management facility which temporarily impounds runoff and discharges it via infiltration through the surrounding soil. While an infiltration facility may also be equipped with an outlet structure to discharge impounded runoff, such discharge is normally reserved for overflow and other emergency conditions. Since an infiltration facility impounds runoff only temporarily, it is normally dry during non-rainfall periods.

1.6 HYDROLOGY

The relationship between rainfall and runoff is modified by development, land use changes, and urbanization. The rainfall values for a design storm event and the intensity-duration-frequency curves for specific recurrence intervals are for all practical purposes constant.

The runoff resulting from similar rainfall events will be modified by antecedent rainfall, time of the year, changes in land use and the amount of paved surfaces. If we look at the impact of paved surfaces in Figure 1.3, it is evident that as the paved area increases, runoff increases and the infiltration decreases. There are several variables in determining runoff from a rainfall event. Antecedent rainfall will fill up depression storage and reduce infiltration and evapo-transpiration. The time of year may cause conditions that will increase runoff such as frozen ground, ice cover, or snow. Snow melt is not a major problem in Hampton Roads, but it does have an impact when it occurs. Construction activity which changes the topography, removes natural swales and depressions, removes tree and heavy forest cover, changes slopes, and introduces drainage systems has an impact by decreasing the amount of pervious area and by reducing the time of concentration in the watershed, both causing an increase in the peak flow.

There are a number of methods that can be used to compute runoff from the

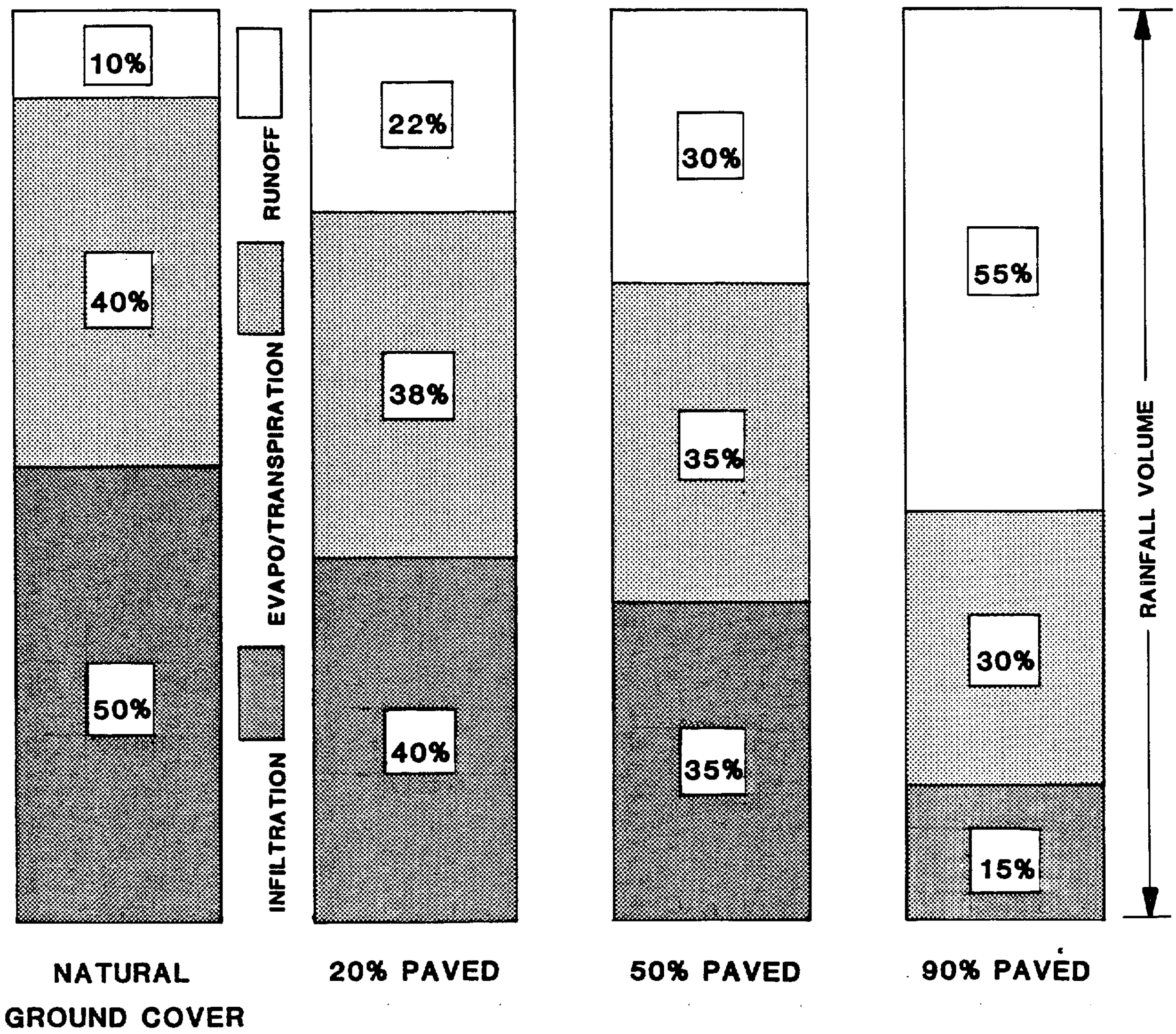

FIGURE 1-3
RUN-OFF FLOWS
RESULTING FROM INCREASED PAVED SURFACE
(Source- Leopold-Hydrology for Urban Land Planning USGS 1968)

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III. The Tidewater Virginia area falls within the generalized Type II storm, except for Virginia Beach which is influenced by coastal events and falls within the Type III category. The difference between Type II and Type III is minor, and Virginia Beach has elected to use the Type II storm for the purpose of standardization. Figure 2-8 shows the SCS Type II storm plotted as a mass diagram where the rainfall depth is plotted against twenty-four hours. As with Figure 2-7, the intensity over any duration of the SCS design storm can be found by the slope of an average line drawn between two points describing the duration around the steepest slope which occurs for this synthesized storm at about 11:30. For example, the intensity for a one hour duration storm event would be found by measuring the depth of rainfall occurring between 11:00 and 12:00 or one hour. That depth would be the inches per hour of intensity for a one hour duration and correlates closely to the intensity for a one hour duration storm of the same recurrence interval from Figure 2-1 and 2-2.

This concept is important to understand because the intensity rainfall value used in the Rational method is the duration value which is selected by computing the time of concentration of runoff in the watershed at the place where the runoff value is being computed. When using the Rational Method in the Hampton Roads area, the intensity value is taken from Figure 2-5 or 2-6.

Several studies have been done to locate the period of most intense rainfall during storm events to make the design storm more closely resemble an actual event. Studies in Chicago, Cleveland, Boston and New York show a consistence when the most intense part of rainfall event usually occurs in the 30-40% time frame of the rainfall distribution. However, the SCS Type II storm follows a more standardized bell-shaped distribution around the twelfth hour. Since the intensity-duration values do not change regardless of when the most intense rainfall occurs, no attempt was made to skew the design storm, shown on Figure 2-7, for the Tidewater Virginia area.

Figures 2-5 and 2-6 show rainfall data for a site specific rainfall gage or station, that being the National Weather Service station at Norfolk International Airport. It has long been recognized that rainfall is variable over a wide area. As the storm front moves, the intensity of rainfall moves. Consequently, the distribution of an equal intensity of rainfall occurring may or may not be widespread over an area. The variation of the depth of rainfall and intensity over the area is referred to as areal distribution. To account for the fact that rainfall may not be occurring over the entire watershed at an equal intensity, Figure 2-9 provides a factor which modifies the intensity based upon watershed size. Figure 2-9 is derived from a variety of sources dating back to the early work of F.A. Marston. This factor is simply multiplied times the site specific intensity value for the selected duration, and then used as the intensity value in the rational method. It should be recognized that the Rational Method should not be used for watersheds over 200 acres.

precipitation data. Many variables are based upon assumptions by the designer or provided by historic data for the specific site under consideration or which can be transferred from similar sites. The methods in general practice include the Soil Conservation Service's TR20 and TR55, the Rational method and variations, and a wide variety of proprietary software programs. Also, the runoff block of the EPA SWMM model is gaining increased usage.

The output of these computations will produce hydrographs of the runoff resulting from a rainfall event under a specific set of circumstances. The first set of circumstances defines the pre-development conditions. The second set defines the post-development condition based on full development. Intermediate circumstances may be important if the development time is going to be lengthy.

1.7 OUTLET AND CHANNEL HYDRAULICS

With the hydrographs developed for pre- and post-development runoff, the type of outlet structure needs to be selected to reduce the discharge flows to the selected value whether it be to maintain a discharge value equal to or less than pre-development flow rates or some lesser value necessary to prevent downstream problems with accumulations due to routing, because of channel limits, or to meet a water quality limitation. There may be cases where the post development discharge may not need to be reduced as low as the pre-development discharge or where it may not be desirable to have a long extended release period. To evaluate the overall impact of the facility under design, a basin or watershed model needs to be used. The purpose of the watershed model is to determine the impact of reduced flows over a longer period of time on discharges from other facilities existing or planned and the other uncontrolled areas of the watershed.

A retention or detention basin reduces the peak flow by storing the water and releasing it at a lower rate for a longer period of time. The actual volume of water is nearly the same with some additional losses due to infiltration and evaporation at the basin. The exception, of course, are those basins designed with no outlet and the only release is through infiltration and evaporation. The analytic technique to move the hydrograph downstream is flood routing. In addition to the outlet structure, the basin will need an emergency spillway to prevent the dam or embankment from being overtopped and susceptible to being washed out causing serious flooding problems. In the streams or rivers, water surface profiles may need to be computed to determine bank full capacity, to establish flooding limits for specific frequencies of flood events, and to obtain velocity data for erosion control practices.

The basin and outlet should be configured to control a range of rainfall events so the overall effectiveness can be increased. It has been found from other studies that an outlet structure designed to provide control from the 2- and 10-year rainfall events is sufficient to provide control from other recurrence frequencies as well. However, local standards, particularly those related to the Federal Emergency

Management Agency, National Flood Insurance program, generally require the specific analysis and outlet sizing for other storm frequencies, such as the 100-year storm.

2.0 GENERAL PLANNING AND ENGINEERING CONSIDERATIONS

This section contains general planning and engineering data and methods which are common to any of the stormwater management practices or facilities. The planning consideration are considered under Stormwater Management Planning. The engineering considerations are considered under the Hydrology and Water Quality Enhancement Sections. The fourth section entitled "Retrofitting" covers the issues which are similar to either retention or detention basins.

The planning discussed in this section is focused on stormwater management facility design. It must be recognized that the comprehensive planning of land use and zoning is also an integral part of total stormwater management. The appropriate use of land development methods, thoroughfare plans, landscaping, open space requirements, cluster development and other comprehensive planning methods and tools need to be part of the total program to reduce non-point source pollution loads and mitigate drainage and erosion problems.

2.1 STORMWATER MANAGEMENT PLANNING

Stormwater Management Planning is done at several levels within the watershed. Starting at the single property site, the planning typically is done by the developers and includes on-site facilities which may incorporate small retention or detention basins, grass swales, infiltration basins or trenches, underground storage and such. These facilities are constructed and generally later operated and maintained by the owner with periodic inspections being performed by the locality. Generally, open on-site storage facilities are incorporated into the landscaping and have only a specific site benefit. The next level involves multiple sites which may be done by the developer and considers a small watershed system. Typically, stormwater management practices or facilities at this level are going to be retention or detention facilities. At this level of planning more than one property owner is usually involved and an agreement must be reached regarding construction and maintenance.

As the size of the system and the number of properties increases, the next level is approached. From the standpoint of a community, this level of planning may be limited by political boundaries; consequently the level of planning is only community-wide. Beyond the community boundary limitations, the stormwater planning is regional or watershed-wide. In the watershed plan, there may be several retention or detention basins which have been planned or constructed as well as other on-site structural BMPs and non-structural methods being used as established by local regulations.

It has been recognized by many planners and engineers that the use of retention and detention basins to serve multiple sites is more efficient and cost effective than several on-site basins serving individual properties. It has also been recognized that the proliferation of basins without watershed or regional planning causes problems just as much as development without any retention or detention basins at all.

2.1.1 ELEMENTS OF THE PROGRAM

2.1.1.1 MULTI-OBJECTIVE PLANNING

As discussed in the introduction, there are multiple benefits which can be derived from retention-detention basins by advanced planning considerations. For the Chesapeake Bay Preservation Areas, the primary objective is water quality protection, whereas in areas regulated by the Stormwater Management Act, the primary objectives are peak flow control and water quality enhancement. A retention or detention facility can satisfy the requirements for both of these Acts plus requirements for erosion and sediment control. With added features incorporating plantings, water quality can be further enhanced, and a wetlands area can be provided for habitat management. The addition of a permanent pool further increases pollutant removal and adds an aesthetic value and possible recreation, especially fishing.

By storing water over a longer period of time than it would be traveling in an open channel, there may be additional infiltration into the groundwater. Typically, this is an advantage; however, care must be taken not to cause an increase in groundwater contamination.

With proper planning, the land surrounding and within the dry areas of the basin can be utilized for other activities such as active recreation, soccer, baseball, softball, football, badminton, volleyball, or passive recreation activities such as picnicking and bird watching.

Multi-objective planning can add many benefits to the community and be a useful tool in quelling the negative comments raised by the local public. Proper design and maintenance will eliminate the complaints often voiced by the public about retention-detention basins, such as child safety, mosquito breeding, aesthetic and health concerns. They can in fact reverse the priority of the objectives in the public's viewpoint of these basins from stormwater management to recreation.

2.1.1.2 OTHER STRUCTURAL ELEMENTS

Retention and detention basins are not the only structural elements that can be part of the plan. The use of grass swales, open ditches, infiltration basins, parking lot storage, roof storage and in-line storage are among the myriad of methods that can be used. Many are discussed in the companion document. Retention and detention basins can be local in nature, that is, on a single site to satisfy the stormwater management for that site or parcel. They also may be part of a system or regional facility serving many sites and larger areas. The location of the basins needs to be considered in a system or watershed planning approach because the hydraulics of the watershed can compound flows or increase the length of time of a higher stage flow, thereby increasing flooding and erosion problems. Secondly, in many areas, there are not many suitable sites that are available for a retention or detention basin. For those watersheds, the sites need to be identified and protected from other uses.

2.1.1.3 NON-STRUCTURAL ELEMENTS

As retention-detention basins are not the only structural solution, structural solutions in turn are not the only element in stormwater management planning. The use of non-structural elements can provide a significant impact on reducing flows and pollution problems and offer the major advantage of not being as expensive to build or maintain. As discussed in the introduction to this section, comprehensive land use planning and the appropriate zoning has the potential to be a very cost effective method in reducing non-point source pollution and mitigation drainage and flooding problems. Non-structural elements include restrictive or creative zoning, use of floodplain zoning and management, and consideration of building code changes. One of the primary building code changes to consider is parking requirements for commercial and industrial development. The number of spaces per unit of development can often be reduced. The use of isolated portions of the parking lot for temporary storage should be considered if it is currently restricted.

2.1.2 LOCATION OF BASINS

Several factors are important in locating detention and retention basins. Several studies and papers have discussed the problems of random multiple basins throughout a watershed. In general, these studies have shown that randomly placed basins can control peak flow from large and infrequent storms, but for the more frequent storms such as 2-year and less, the impact is only seen immediately downstream of the facility, with occasions where the impact causes greater peak flows as other sub-areas join and the hydrographs accumulate, thereby compounding downstream flooding. This points to the need for watershed master planning to locate basins within the watershed. Generally, basins used to control peak flow are best placed in the upper reaches of the watershed rather than in the downstream areas. However, the shape of the basin, the need to reduce peak flows in downstream reaches, and the aggregate effects of lengthened discharge hydrograph must be considered. Retention and detention basins are usually used on sites with a watershed of ten acres or more. This is not a hard and fast rule, but cost comparisons with other methods usually show other on-site structural or BMP methods to be more cost-effective.

A specific basin location needs to consider the community. The basin must become part of the landscape and be aesthetically pleasant. Topography, soils, environmentally sensitive areas or areas of cultural importance need to be considered. The objective of the basin becomes important. If multi-objective uses are being planned, then the opportunities for recreation, wildlife habitat management, and water quality enhancement need to be considered. The basin needs to be structurally safe, the embankment needs proper engineering, and methods to by-pass rare storms need to be designed into the structure. The design needs to consider protecting people from harm by proper design of side slopes and permanent pools. The outlet structures need to be designed to protect people from getting hurt. Fences and signs have limited value. Better design features include minimizing the visibility, putting the outlet structure away from shore, using trash racks and extending them totally over the outlet are some methods. The Task Committee report "Stormwater Detention Outlet Control Structures," ASCE, New York, 1985, is a good reference.

2.1.3 BMP SELECTION

Selecting the appropriate regional facility depends upon the objectives and the location. Ownership and maintenance should be a consideration, because if there is any chance that maintenance will not be performed, it should not be built. This fact in itself points heavily to the need for municipal ownership (or easement) and maintenance, or at least a method of routine inspection and enforceable regulations and ordinances.

Table 2-1 compares the multiple objectives discussed earlier with the detention and retention basins with the various modifications and provides a relative value of meeting the objective opportunities. The opportunities for meeting the objective increase as the modifications are added. Certain modifications such as extended release and water quality storage may required if the basin is in a locality regulated by the Virginia Stormwater Management Act or the Chesapeake Bay Preservation Act. A review of Table 2-1 would indicate that the best solution for a multi-purpose or multiple objective basin is a retention basin with extended release, the added water quality storage, and a wetlands fringe, and in general, it is most likely the best solution if the space is available, funds are available, and if in fact all of the objectives need to be met. In some areas, there will not be space for a permanent pool. In some areas, there may not be the need to consider aesthetics, recreation, or habitat protection or management. For example, small retention or detention basins may not suitable for these added features. Basins in heavy industrial areas or other areas of limited public use need a different evaluation. In order to provide data on typical cost and land area requirements, several scenarios were developed as part of this work by URS Consultants, Inc. Figure 2-1 shows typical land requirements in an area with Hampton Roads' topography for the various types of facilities. The size for a retention or detention facility varies from roughly 2% to 5% of the watershed area. This is based on basins averaging 6' deep and allows area for access roads and a buffer strip. In a multipurpose basin, additional land has been allowed for buffer, recreation, and a permanent pool to meet the water quality volume requirement. Three different development scenarios were used for Figure 2-1 of 200, 500, and 1000 acres.

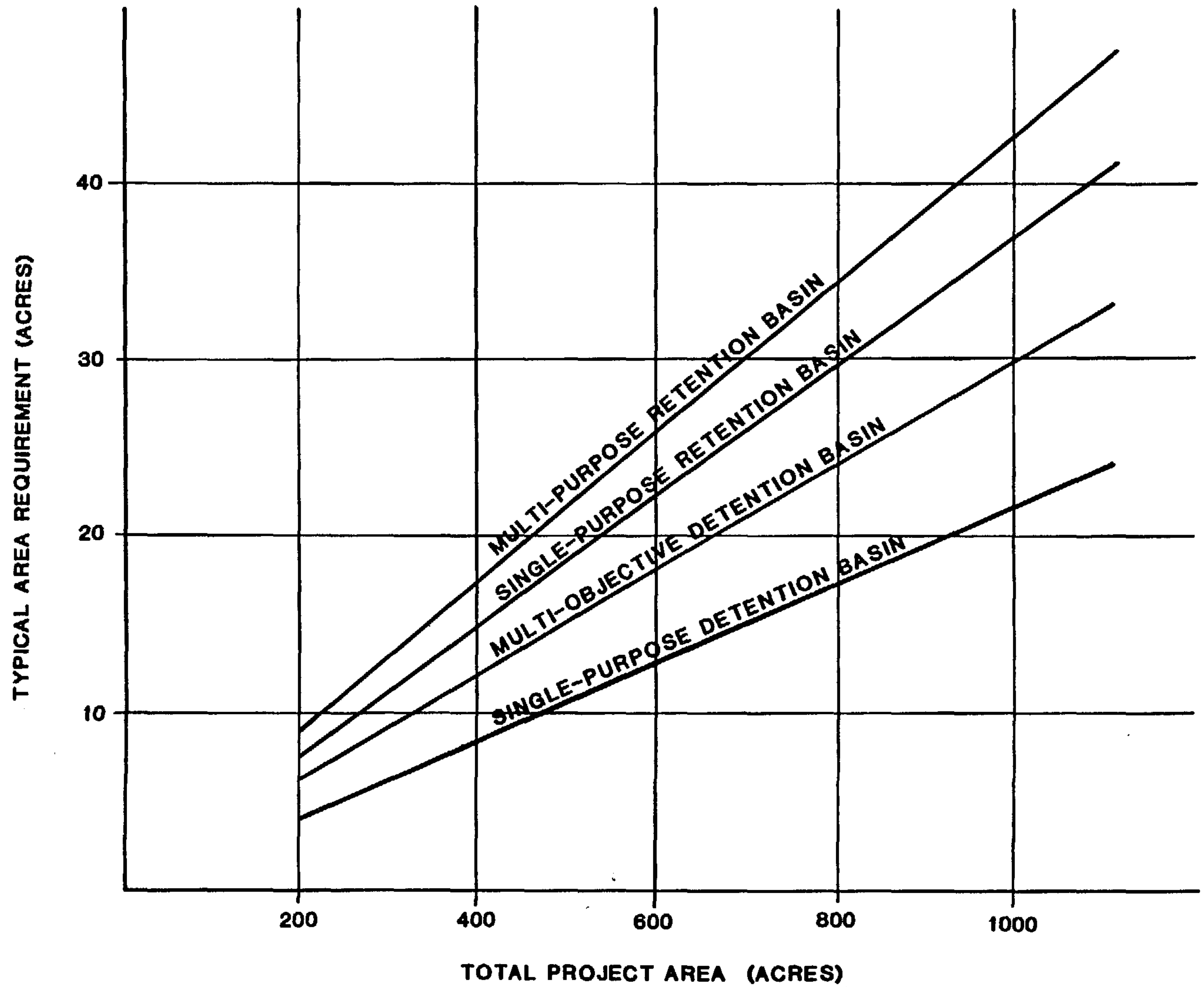

FIGURE 2-1

STORMWATER FACILITY AREA VS. TOTAL PROJECT AREA

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Cost curves shown for some typical BMPs in Figures 2-2 to 2-4 compare various methods of managing stormwater for a range of watershed or project areas. Figure 2-2 is for an infiltration trench, Figure 2-3 is for a typical swale design, and Figure 2-4 is for an oil-water separator. These were based on a design of a facility to control a 10-year runoff from areas with C-factors as shown. The designs were based upon typical standards and an assumed lot development of the different acreages of 1,3, and 5 acres.

If the water quality performance criteria of the Chesapeake Bay Preservation Regulations can be met in other ways, the basin modifications can be simplified. The end result should be a cost-effective design to meet the specific objectives at that site and the regulatory criteria established.

TABLE 2-1

SELECTION EVALUATION OBJECTIVES

	Peak flow reduction	Water quality enhancement	Aesthetics	Recreation	Habitat protection	Habitat management	Groundwater recharge
Detention Basin							
Basic Basin	1	4	4	1	4	4	4
add extended release	1	3	4	1	4	4	3
add wetlands bottom	1	3	2	1	3	3	3
Retention Basin							
Basic Basin	1	4	3	1	3	3	3
add extended release	1	3	3	1	3	3	4
add water quality storage	1	2	2	1	2	2	4
add wetlands fringe	1	2	1	1	1	1	4

1 - Excellent
2 - Good
3 - Fair
4 - Poor

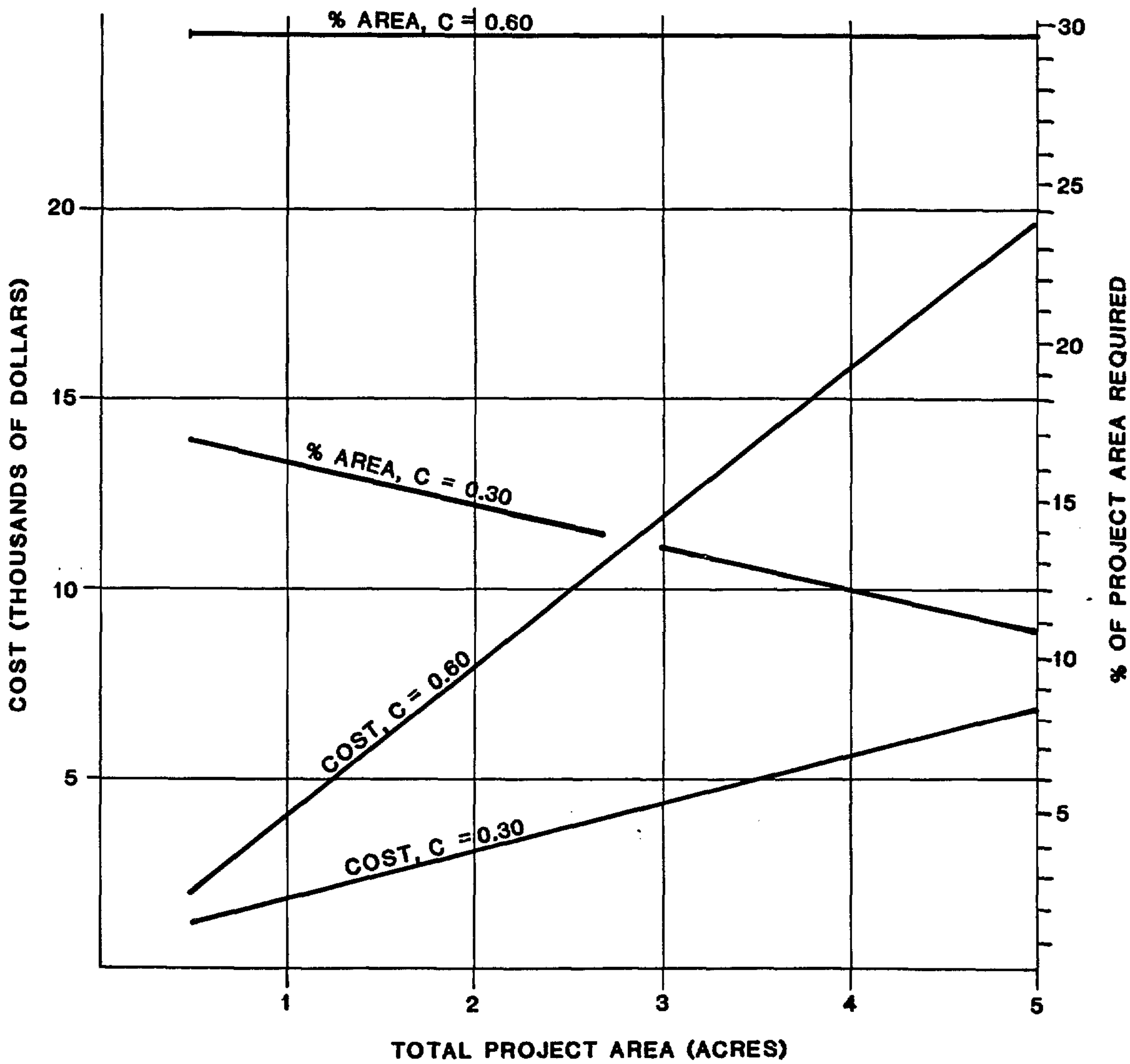

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FIGURE 2-2
INFILTRATION TRENCH (WITH 20' FILTER STRIP)

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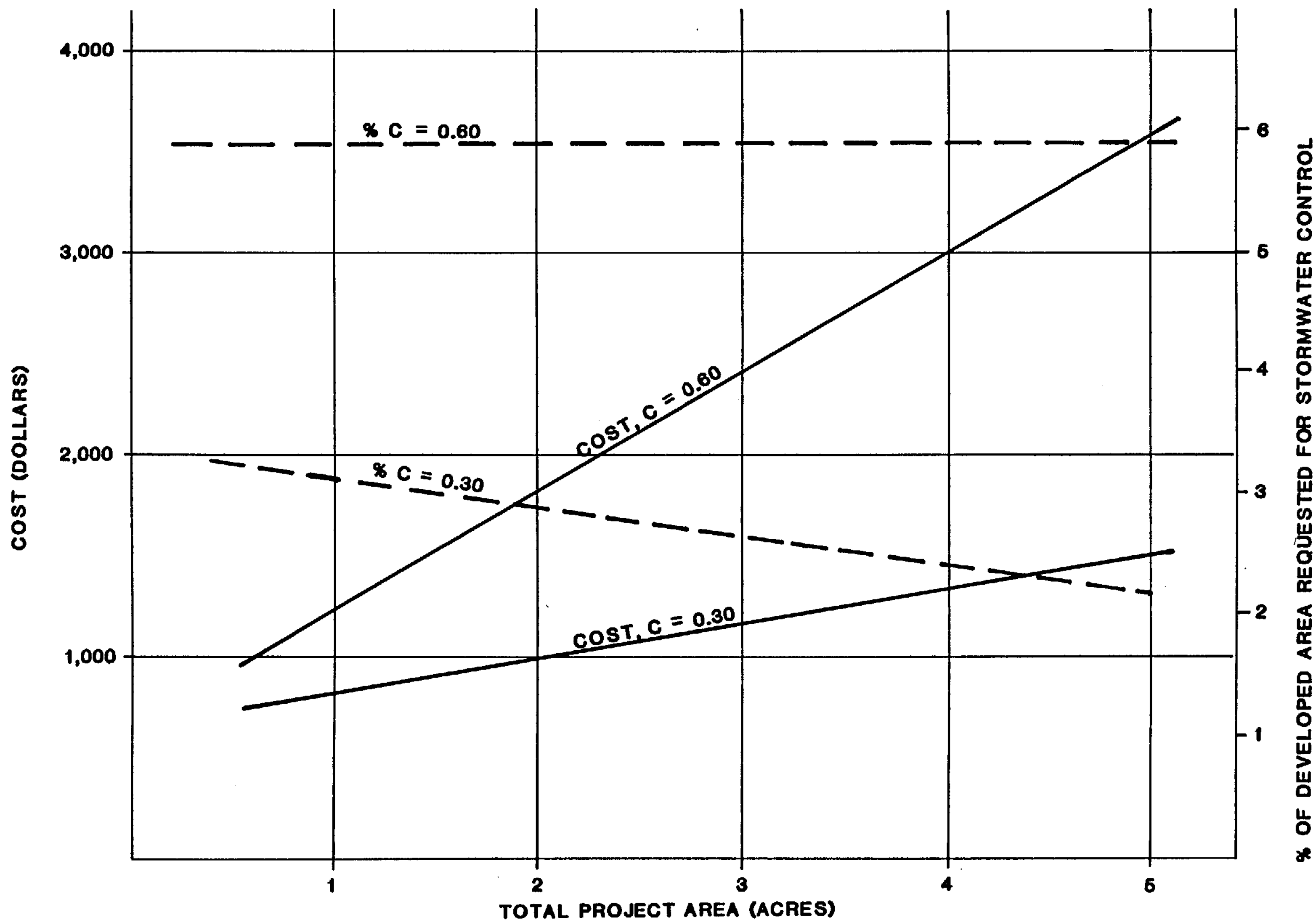

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FIGURE 2-3
SWALE FOR 2 & 10 YEAR STORM

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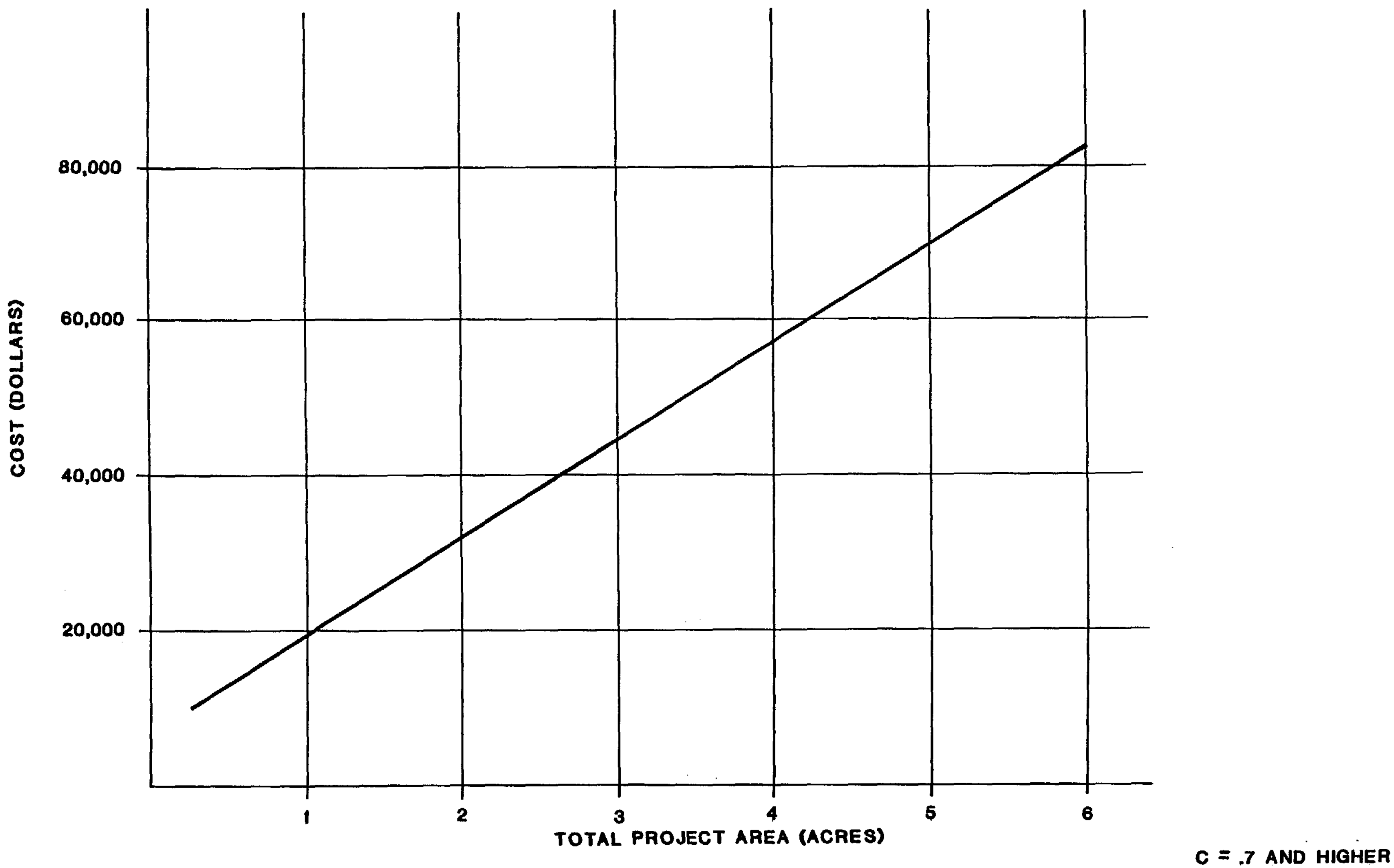

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FIGURE 2-4
GRIT-OIL SEPARATOR FOR HIGHLY DEVELOPED INDUSTRIAL SITES

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2.2 HYDROLOGY

In stormwater management, the aspect of hydrology we are most concerned with is the rainfall and subsequent runoff. Rainfall data is available from NOAA, National Weather Service and has been statistically synthesized into events of specific return intervals. The runoff resulting from precipitation has been measured by using gaging stations along major water courses. A variety of methods are available to estimate or account for the difference between the total rainfall and that which runs off. These methods allow computation of the peak flow and many will generate data for hydrograph development. Stormwater drainage systems in the past have basically dealt with the peak flow; however, with stormwater management practices, it is necessary to develop a hydrograph so that the volume of runoff is known. With the design of permanent pools in retention basins, we also need data on base flows.

2.2.1 RAINFALL DATA

The rainfall data is commonly described in an Intensity - Duration - Frequency (IDF) curve which is available for site specific rain gages throughout the area from the National Weather Service. The Norfolk IDF curve is shown on Figure 2-5. This data has been plotted for a 6-hour duration on Figure 2-6. One of the important characteristics to recognize in these two curves is that the time value is the duration of a rainfall event at an intensity equal to or greater than the number derived by selecting an intensity from the frequency curve. The time value has no relationship to the time from the beginning of the rainfall event. If a value of intensity is read for a duration for a specific frequency, it means that from the rainfall data collected over the period of record that specific intensity over that duration of time has occurred equal to or greater than that value that frequently. Although common practice is to discuss frequency as a yearly event, that is, once in five years or once in fifty years, it is statistically more appropriate to use a percentage of recurrence interval. A five year storm event has a 20% chance of occurring in any one year. A fifty year frequency has a 2% chance of occurring in any one year.

The National Weather Service has taken the site specific rainfall data and generated a variety of data presentations useful for large scale planning, but for the purposes of stormwater management planning at this level, Figures 2-5 and 2-6 provide the needed basic data. Variations of Figure 2-5 include rearranging the data to create a design storm where the time scale is time from the beginning of the rainfall event. The purpose of this exercise is to obtain values of antecedent rainfall and to be able to compute hydrographs that show the impact of long duration storms and allow volumes of these storms to be computed. Figure 2-7 shows a six-hour design storm for Norfolk. In this figure, the rainfall depth has been plotted for a six-hour storm for the frequencies of return intervals shown. The Soil Conservation Service has developed typical design storms for the United States. These storms have been categorized as Type I, Type IA, Type II, and Type

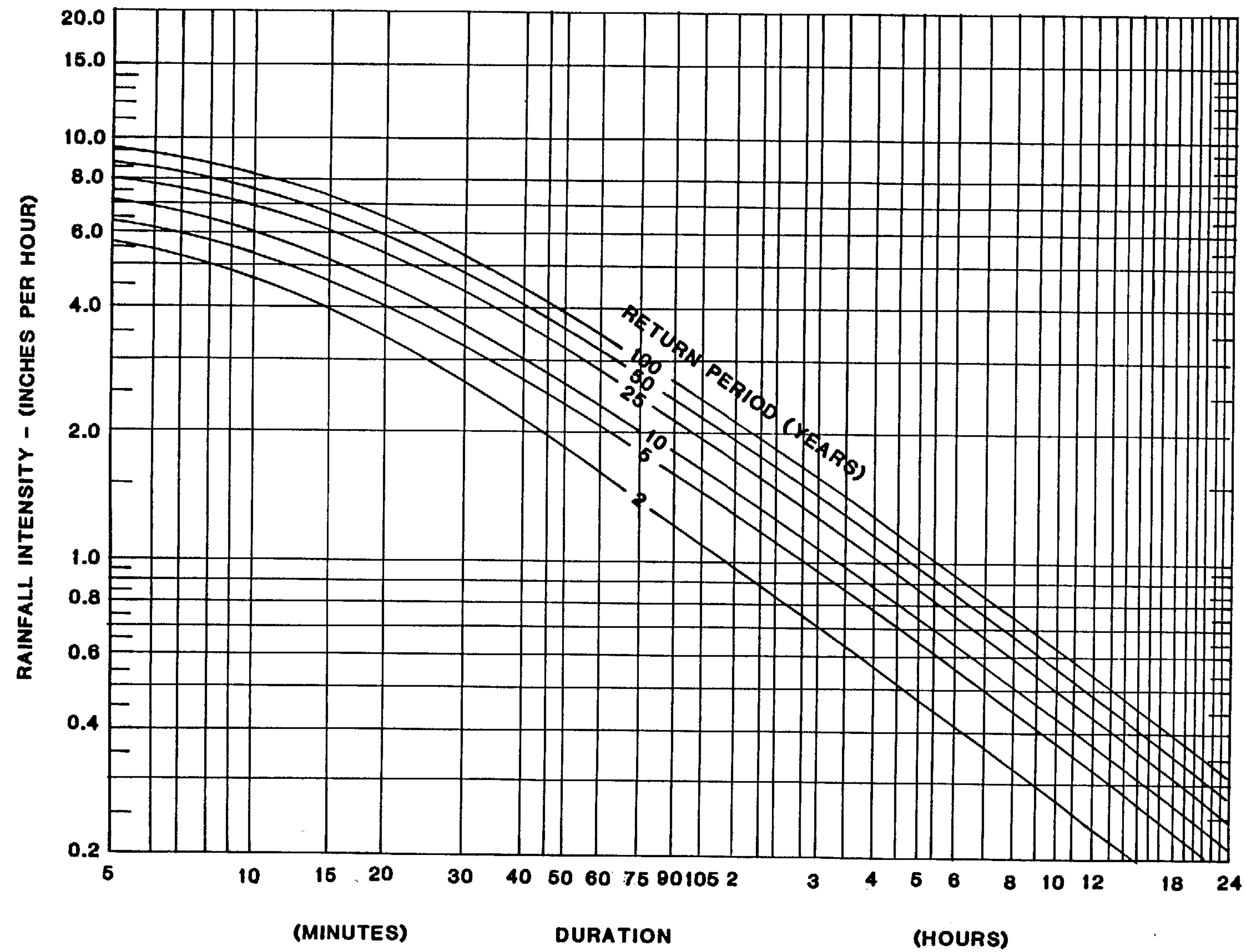

FIGURE 2-5

INTENSITY DURATION FREQUENCY CURVE-NORFOLK, VA

(Source- NOAA Technical Memorandum NWS HYDRO- 35, June 1977)

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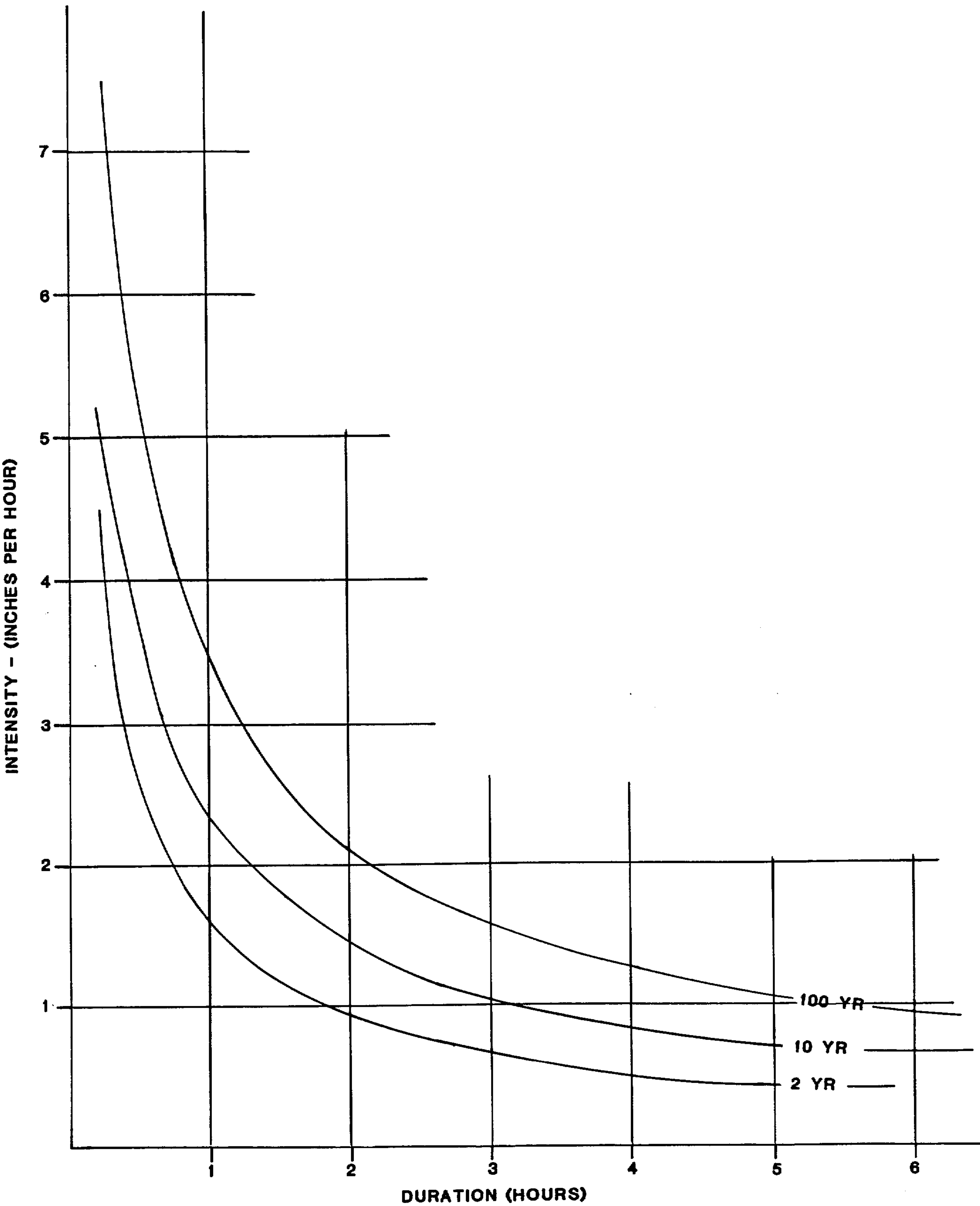

FIGURE 2-6

INTENSITY DURATION FREQUENCY CURVE–
NORFOLK,VA (0–6 HOURS)

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(Source– NOAA Technical Mememorandum NWSS Hydro–35, June 1977)

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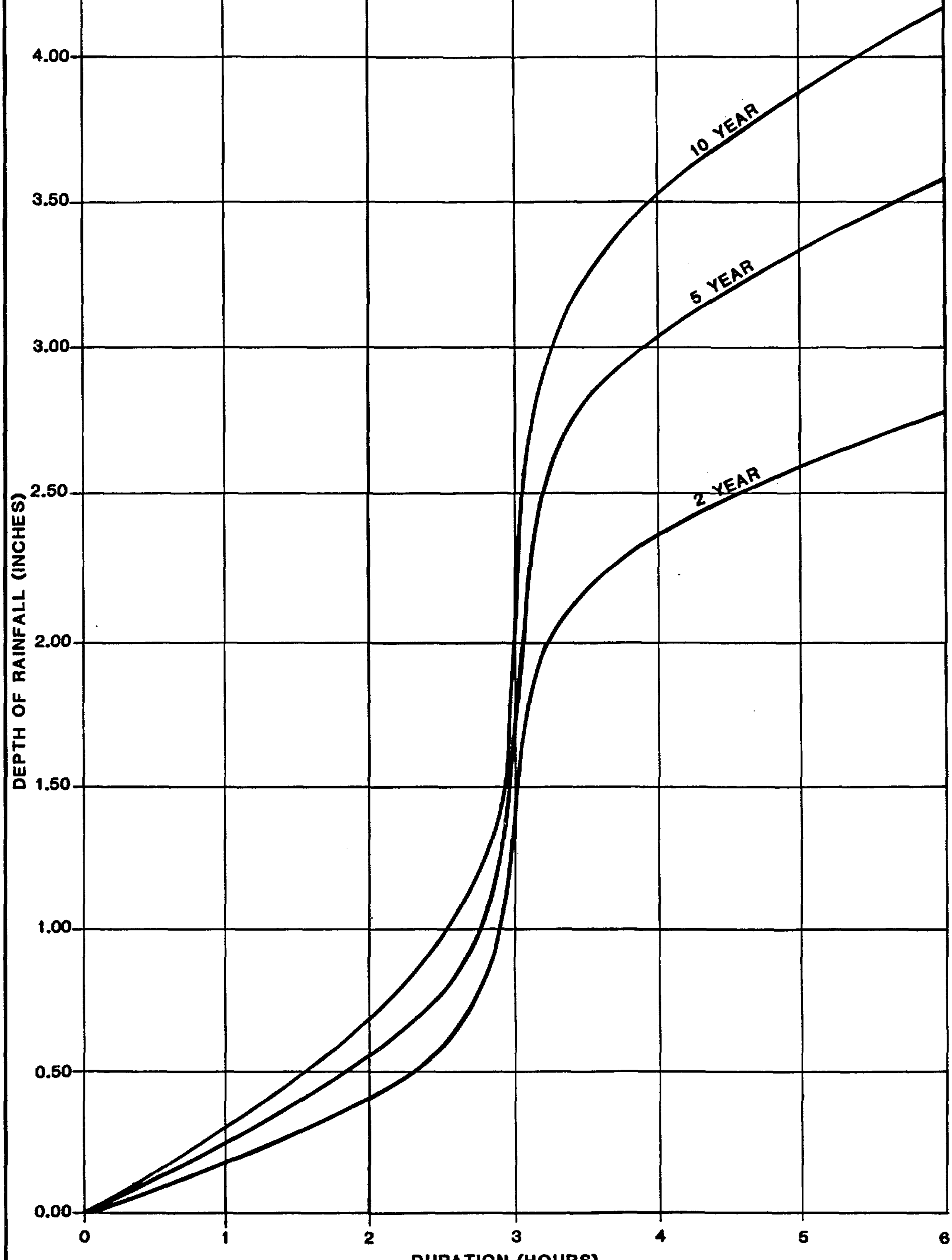

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FIGURE 2-7

DESIGN STORM MASS DIAGRAMS

NORFOLK, VIRGINIA

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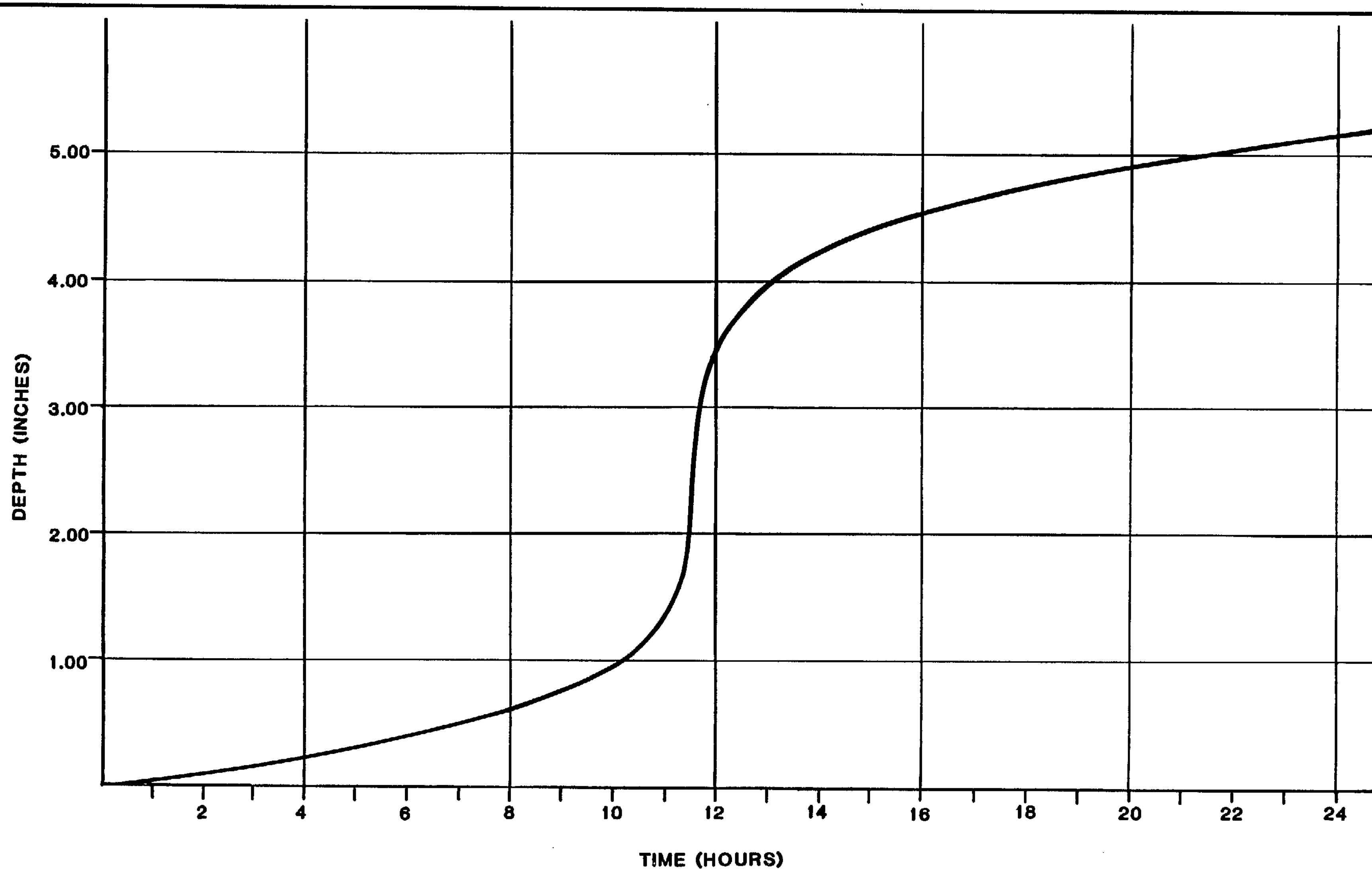

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FIGURE 2-8
DEPTH VS. 24 HOUR STORM, SCS TYPE II, 10-YEAR

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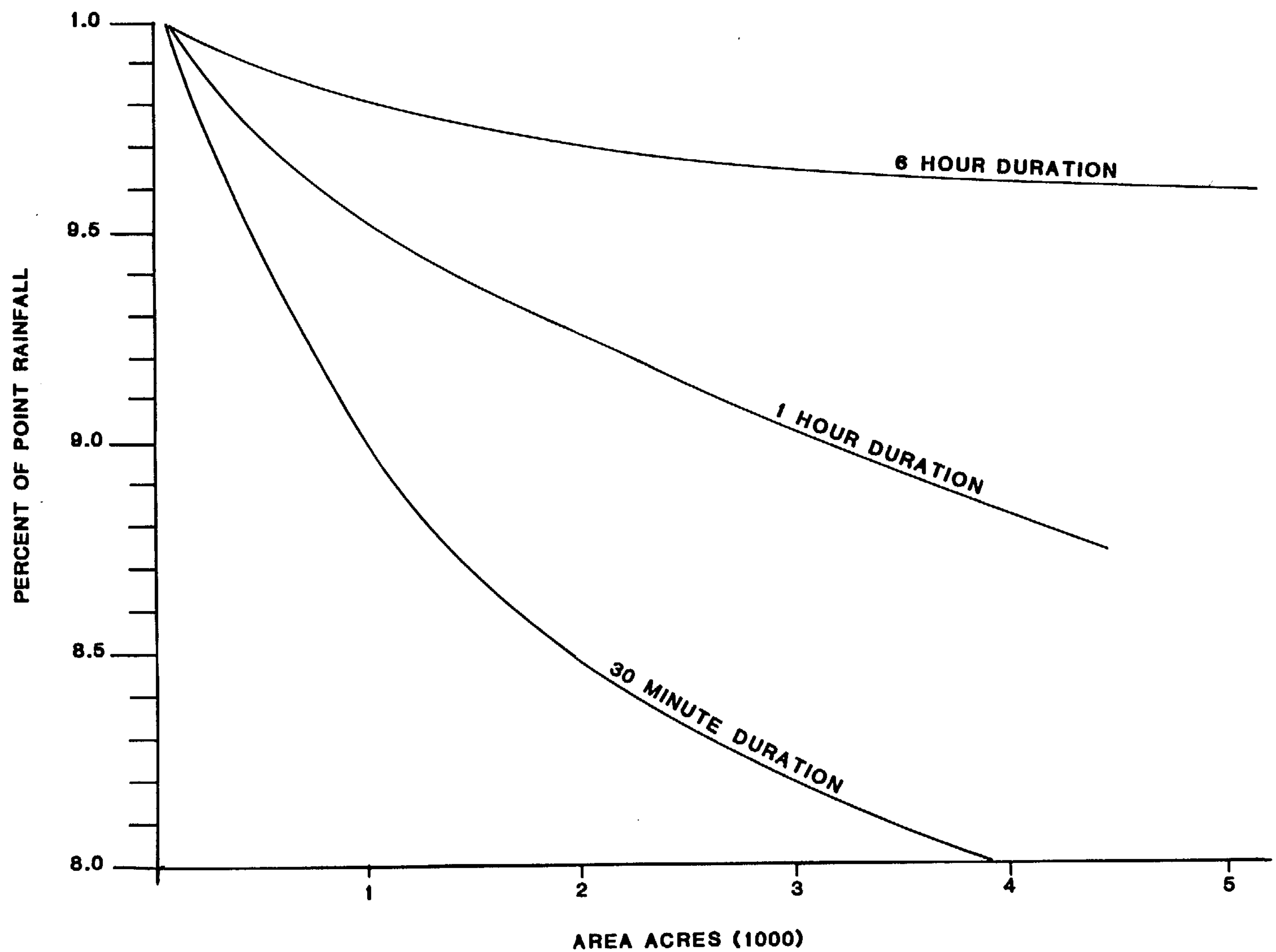

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FIGURE 2-9
RATES OF OVERALL RAINFALL TO MAXIMUM POINT RAINFALL

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2.2.2 HYDROGRAPHS AND PEAK FLOW COMPUTATIONS

The design of retention and detention facilities requires the development of hydrographs to arrive at volumes of water running off the watershed and the peak discharge of that runoff. The pre-development hydrograph becomes the measure of performance because it defines the peak flows for the two- and ten-year frequency storms which may need to be maintained. The post-development hydrograph before the stormwater management practices are put in place defines the additional volume of runoff and the increase in peak flow. The post-development hydrograph becomes the inflow hydrograph for the retention - detention facility after any non-structural or on-site BMPs have been accounted for.

Routing the inflow hydrograph through the retention - detention basin by considering the storage elevation curve and the outlet hydraulics will produce the outflow hydrograph. The retention - detention basin storage elevation curve is governed by the topography of the site and height of the impoundment. This curve is developed by plotting the available flood storage against the corresponding pool elevation. The storage is determined by measuring the surface area flooded at contour elevation intervals and computing the volume between the intervals.

The hydraulics of the outlet or principal spillway control the discharge. Since the hydraulic head over the outlet structure is the primary energy source, the pool elevation can be related to the hydraulic head over the outlet structure and a corresponding outflow-elevation relationship can be developed. The type of outlet structure and size will provide the other design data to compute the discharge with the given elevation head.

By selecting the appropriate outlet structure type and size and with sufficient storage, the outflow can be controlled to limit the two- and ten-year runoff events resulting from development to those that existed prior to development, or to those established by watershed/regional SWM plans, thereby satisfying the requirements of the Virginia Stormwater Management Regulations and the Erosion and Sediment Control Regulations with regard to peak flow control. It should be noted that the CBPA regulations require that post-development runoff pollutant loads not exceed pre-development runoff pollutant loads.

There are a number of methods which can be used to compute peak flows and to develop hydrographs:

Rational Method: The Rational Method is widely used for computing peak rates of runoff from areas generally less than 150 to 200 acres. Virginia Department of Transportation (DOT) allows the Rational Method for watersheds up to 200 acres. Others have limited it to watersheds as small as fifty acres, but general agreement is found in the 150-200 acre range. The method produces the maximum discharge from a given uniform rainfall event when the entire watershed is contributing runoff to the outflow at the point of design. In order for the entire watershed to contribute, the uniform rainfall event must occur for a length of time that it takes water to flow from the most remote part of the watershed to the point of design. This time is called the "time of concentration." The intensity value is selected from the Intensity - Duration - Frequency curve for the selected frequency of storm for the duration of time which is equal to the time of concentration.

Table 2-2 shows typical values of "C" to be used in the Rational Method for a variety of land uses. These factors do not correlate exactly with the percentages of impervious area because they have been increased to account in a general fashion for slope, infiltration, and intercepted flow. Table 2-2 was generated from a wide variety of sources, and the present impervious data are averages computed for a comparison.

TABLE 2-2

TYPICAL "C" COEFFICIENTS FOR RATIONAL METHOD

Land Use in Area	Runoff Coefficients	Percent Impervious
Business		
Downtown areas	0.70-0.95	90%
Neighborhood areas	0.50-0.70	60%
Shopping Malls		
Regional - enclosed	0.65-0.85	
Strip Malls	0.70-0.90	
Business Parks	0.60-0.75	
Industrial Parks	0.65-0.80	
Residential		
Single Family Suburbs	0.30-0.50	20-25%
Single Family Urban	0.40-0.60	30-35%
Multiunit detached	0.40-0.60	
Multiunit attached	0.60-0.75	75%
Planned Unit Development	0.30-0.50	20-30%
Apartments - Urban	0.50-0.80	75%
Industrial		
Light areas	0.50-0.80	
Heavy areas	0.60-0.90	
Parks, cemeteries	0.10-0.25	15%
Playgrounds	0.20-0.35	20%
Railroad yard areas	0.20-0.40	
Unimproved areas	0.10-0.30	
Streets		
Asphaltic	0.70-0.95	
Concrete	0.80-0.95	
Brick	0.70-0.85	
Drives and walks	0.75-0.85	
Roof	0.75-0.95	
Lawns; Sandy Soil:		
Flat, 2%	0.05-0.10	
Average, 2-7%	0.10-0.15	
Steep, 7%	0.15-0.20	
Lawns; Heavy Soil:		
Flat, 2%	0.13-0.17	
Average, 2-7%	0.18-0.22	
Steep, 7%	0.25-0.35	

In order to select the appropriate "C" factor, typical percentages of impervious area of the entire area should be measured. The placement and contours of pervious areas needs to be considered. If the pervious areas drain to low pervious swales or ditches, the runoff will be contained. If curbs separate the impervious areas from higher pervious islands or median strips, the pervious areas will have less impact on reducing runoff. Areas with higher slopes will have a higher runoff "C" value. Pervious areas that are grass covered will hold more water from running off than bare soil, and the type of soil has an impact. All of these factors need to be considered when selecting a "C" factor. For final design, a field visit to the site and watershed along with a study of recent aerial photographs is a significant help in making the estimate.

For small watersheds less than 160 acres, the Rational Method has been used to develop hydrographs. The two methods most commonly used both make assumptions that the hydrograph is triangular. The method developed by A.S. Paintel assumes the hydrograph shape has a rising limb equal in time to the time of concentration and the recession limb of the hydrograph equal to 1.5 times the time of concentration. The second method often used merely assumes the falling limb is an image of the rising limb in a triangular fashion. Both of these methods need to check volumes of runoff by calculating the impact of longer duration storms of the same return interval to verify the maximum storage requirements. Figure 2-10 illustrates the concepts. When the discharge is computed for durations other than the time of concentration for storms of a like return interval the simplified hydrograph takes the shapes as shown in Figure 2-11. By computing the volume of total runoff for a variety of storms with varying durations of the same return interval a storage curve can be plotted which will show the maximum storage volume required for that specific site for the selected "C" values for the selected rainfall intensity frequency curve or return interval.

This storage curve can be used to find the required volume for a variety of outfall discharges by plotting the discharge - duration curve as shown on Figure 2-12.

<u>Modified Rational Method</u>: The Virginia Department of Transportation has modified The Rational Method by adding a correction factor to account for the influence of antecedent rainfall. As discussed earlier, the duration factor in the selection of the intensity has no relationship to the time from the beginning of rainfall. The design storm concept further illustrates this fact and in order to account for reduction in infiltration, transpiration, evaporation, and depression storage, VDOT has derived a correction factor for storms with a frequency of greater than 10 years as shown in Table 2-3. This factor is multiplied times the discharge computed using the regular Rational Method.

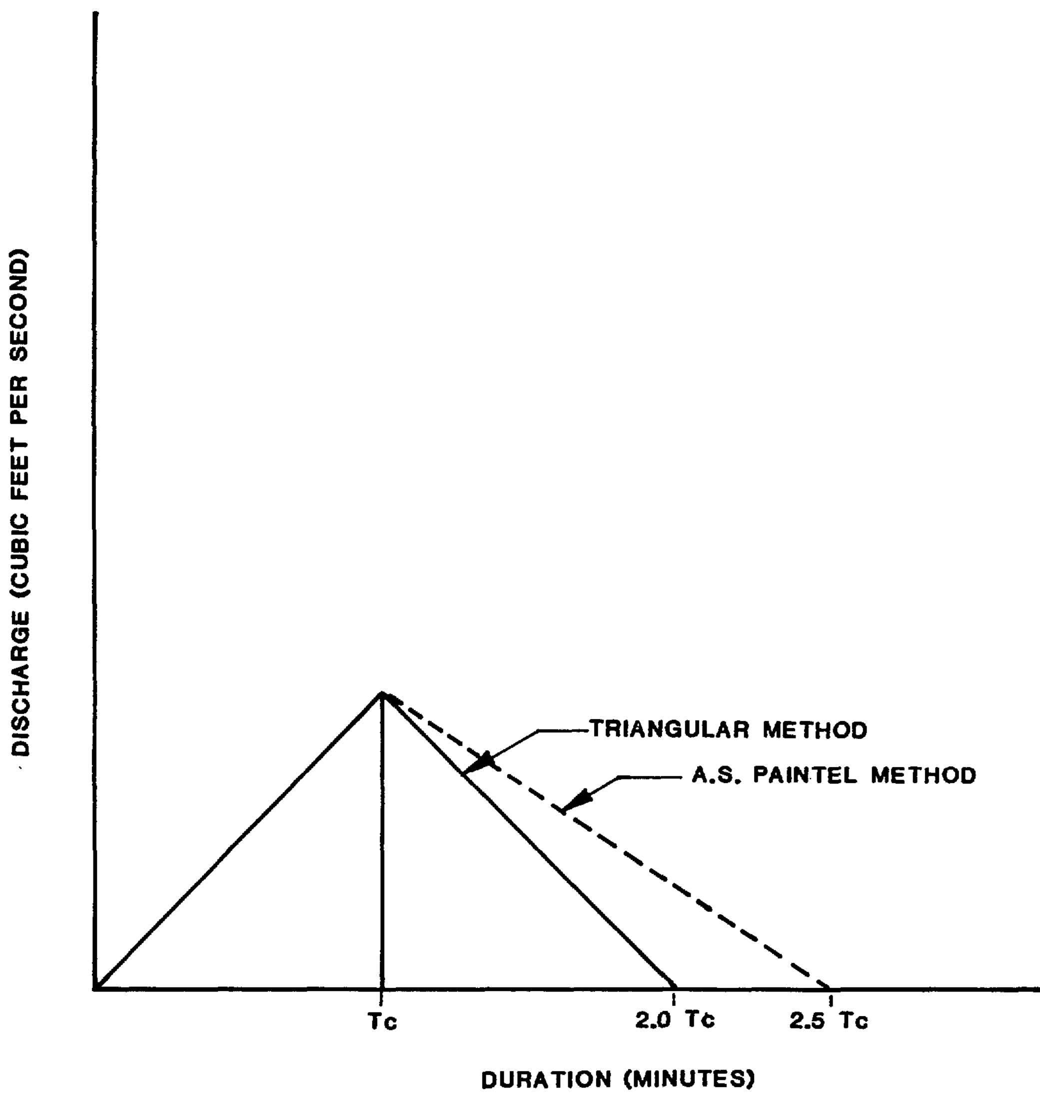

FIGURE 2-10
TYPICAL ASSUMED HYDROGRAPHS
USING RATIONAL METHOD

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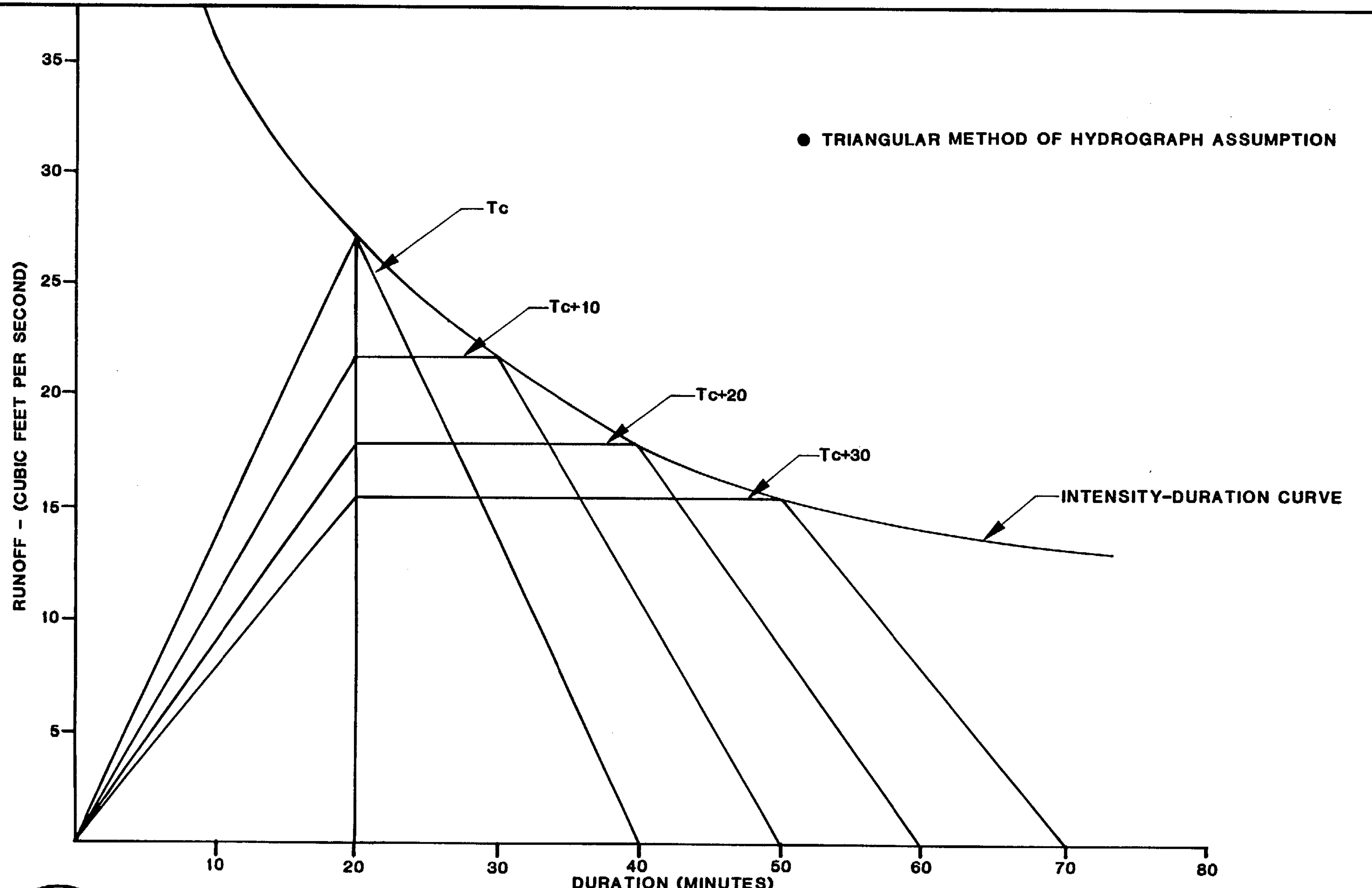

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FIGURE 2-11

POST-DEVELOPMENT STORAGE VOLUME CALCULATION

USING THE RATIONAL METHOD CONSIDERING THE TIME OF CONCENTRATION (NOT CONSIDERING DISCHARGE)

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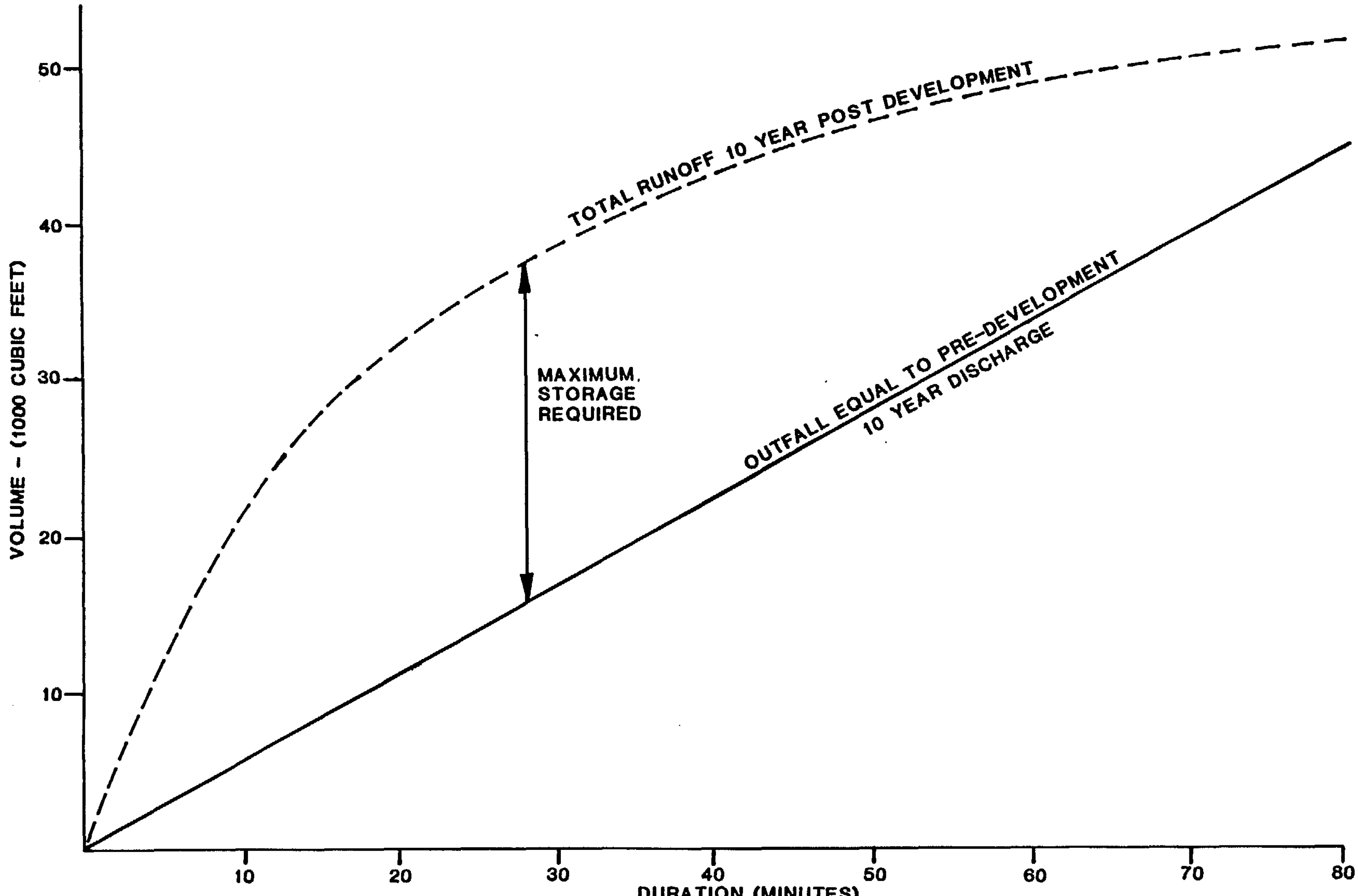

FIGURE 2-12

DISCHARGE - DURATION CURVE

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TABLE 2-3

MODIFIED RATIONAL METHOD
CORRECTION FACTOR

Frequency or Recurrence Interval	Correction Factor
10 years or less	1.00
25 years	1.10
50 years	1.20
100 years	1.25

The Anderson Formula: The VDOT Manual includes a method described as The Anderson Formula. This method was developed for use in Northern Virginia and because of the localized data analyzed to arrive at the formulation, it should not be used in the Hampton Roads area. This method is generally used on watersheds greater than 200 acres.

Snyder Method: The Snyder Method is described in the Virginia Department of Transportation as another method of computing peak flows. It is valid throughout Virginia, and is generally used on watersheds greater than 200 acres.

SCS Technical Release 55 and 20: TR55 can be used to compute peak flows, generate hydrographs, and perform routing. However, when routing through a pond or reservoir the TR55 methodology is an approximation. It is suitable for planning but not for final design. TR20 should be used for final design. TR20 allows the direct input of the outflow hydraulic characteristics from the outflow-elevation curve.

EPA SWMM - Runoff Block: This method may be used to compute stormwater runoff. The most recent version allows continuous simulation and also will estimate pollution loads. The program is complex and requires experience.

Other Methods: There are a number of other methods which can be used; however, since the staff from localities and the state do not have access to all of the proprietary software, it is incumbent upon the designer to supply sufficient detail for the local or State staff to check and verify the design calculations. A checklist has been provided in Appendix A as a guide to the material and data needed.

2.2.3 LOW FLOW/BASE FLOW

When a permanent pool is established, the base flow and runoff from frequent rainfall events need to be sufficient to maintain the pool. Evaporation, transpiration, and infiltration losses must be overcome. Evaporation is the greatest in the months of April through November as is transpiration. Infiltration losses occur constantly if the basin bottom is above the groundwater table; however, infiltration losses should decrease over time as the sediments seal off the bottom. Infiltration can also be reduced by use of a clay or geotextile liner.

Rainfall occurs on an average of once every three days in the Hampton Roads area. These frequent rainfalls are usually of a low intensity lasting for several hours which result in a low total depth of precipitation. Generally, for low-intensity, low-duration storms, only the rainfall that falls on impervious area directly connected to the stormwater drainage system will become runoff. The designer needs to compute the expected volume of runoff from the impervious area directly connected per month and compare that to the volume in the permanent pool and the expected evaporation from the pool surface area. Evaporation for this area is shown on Table 2-4 along with monthly rainfalls data.

TABLE 2-4

PRECIPITATION AND EVAPORATION DATA
Norfolk, Virginia, 1948-1990
(Source: NOAA - Climatic Summary of the United States)

Month	Days of rainfall greater than a trace	Normal Precipitation	Minimum	Evaporation
J	10.4	3.72	1.05	--
F	10.4	3.28	0.86	--
M	11.0	3.86	0.75	--
A	10.1	2.87	0.43	6.42
M	10.0	3.75	1.41	6.98
J	9.3	3.45	0.37	7.73
J	11.2	5.15	0.77	7.69
A	10.5	5.33	0.74	6.60
S	7.8	4.35	0.26	4.92
O	7.6	3.41	0.57	3.57
N	8.0	2.88	0.49	2.53
D	9.0	3.17	0.67	--
ANNUAL	115.3	45.22		

*Evaporation is measured at Holland, Virginia, and the period of record is 11 years.

In normal months, the precipitation will be sufficient to maintain a permanent pool if infiltration is insignificant. However, the average basin will lose permanent pool storage in the months of minimum precipitation. Based on a very rough calculation, a basin with a contributing watershed of about 35% impervious surface will not be able to maintain a permanent pool if the monthly rainfall is less than 1.5 inches per month. This situation can be expected to occur about once every five years in the summer months when evaporation is the highest and the situation would be the most critical. A study of drought flows would better these values. Statistical data on minimum rainfall is not readily available.

2.3 WATER QUALITY ENHANCEMENT

Water quality can be enhanced in a retention or detention basin by increasing the time the water is stored under quiescent conditions. The sedimentation of total suspended solids also removes other pollutants such as lead, zinc, copper, and some organic priority pollutants. Further enhancement is found when a permanent pool is established that has a storage time in general greater than 24 hours. More recent studies have shown that additional removal of pollutants, specifically nutrients, soluble phosphorus nitrates and nitrates can be accomplished by aquatic plants to grow in the permanent pools or in shallow marshes. There are many variables in determining the efficiency of a basin to remove pollutants such as particle size distribution, initial concentration, pH of the water, and configuration of the basin.

The concentration of pollutants will vary with the time between rainfall events, character of the area, and degree of air pollution.

2.3.1 METHODS

Several studies have generated typical ranges of efficiencies for retention and detention basins for various pollutants, and this data has been summarized in Table 2-5. These are expected removal rates for a basin where the pollutant stormwater concentrations are typical of the NURP data, and the basins are configured for effective removal consistent with standard design parameters.

In basic detention and retention basins, the ability of the basin to capture sediment is referred to as the trap efficiency.

TABLE 2-5

POLLUTANT REMOVAL EFFICIENCIES

	Percent Removal				
	TSS	TP	Heavy Metals	Organics	N
Detention Basin					
Basic (6 hr storage)	30-60	20-50	25-85	30-60	
extended release (30 hr storage)	60-90	20-60	60-85	60-90	
wetlands bottom		60			
Retention Basin					
Basic	30-60	35-65	25-85	30-60	
extended release	60-90	30-70	60-85	60-90	
water quality storage		65			
wetlands fringe					

TSS - Total suspended solids
TP - Total phosphorus
Organics - Proportional to TSS
Heavy Metals- Median EMC concentration value from NURP
N - Nitrogen

The percent removals shown have been obtained from several sources and show removal efficiencies within the range one could expect. The data was compiled from many types of basins, and removal efficiencies recommendations are not consistent in the literature. When used for a preliminary design, the lower numbers should be used. Actual design values should be computed using 1986 EPA recommendations or other similar methods.

2.3.2 REGULATORY REQUIREMENTS

In Virginia, the State Stormwater Management Regulations require a thirty hour release from detention basins of the water quality volume of 0.5 inches of runoff and permanent pool storage in retention basins of 1.5 inches of runoff from the project area, or 3 times the water quality volume.

The Chesapeake Bay Preservation Act establishes a performance criteria for the selected keystone pollutant, Total Phosphorus. These are discussed in Sections 1.3.1 and 1.3.2. The methods to provide these requirements are discussed in Sections 3 and 4 separately.

2.4 RETROFITTING

Retrofitting existing stormwater management facilities that were designed for the single purpose of drainage or flood control into a facility that will improve water quality to some degree can generally be accomplished without major difficulty.

Retrofitting existing detention and retention basins by extending the release period for frequent rainfall events can be done by modifying the outlet. Adding a wetlands bottom or wetlands fringe can be done with only minor construction. Detailed discussion of retrofitting can be found in Sections 3.4.3.3 and 4.4.3.3 of this manual. A detailed analysis needs to be done to make certain the modifications do not impact storage requirements; however, usually any extension of the storage time will result in a water quality improvement.

3.0 DETENTION BASINS

3.1 DESCRIPTION

Detention basins temporarily store stormwater runoff and discharge it to the downstream conveyance system through an outlet structure designed to completely empty the facility over a relatively short time period, usually six hours or less. In their basic form, they have historically only controlled stormwater quantity, but recent studies have shown that extending the stormwater release time to 24 to 30 hours can significantly enhance stormwater quality, making them suitable for Best Management Practices of stormwater management.

3.2 APPLICABILITY

Detention basins are applicable for controlling the quantity and quality of runoff from residential, industrial, and commercial developments, highways, or other areas of urbanization where excess runoff must be detained and released at controlled rates so that discharges and pollution levels are maintained within the capacities of existing downstream systems and do not exceed pre-development levels.

Detention basins can be designed to control runoff from an individual development site, multiple development sites, or entire drainage areas. Regional planning, as described in Section 2, is the best method of selecting locations for basins that will serve more than one development site. It has been shown in other studies that individually designed and randomly located basins may actually create or exacerbate downstream flooding by the combination of discharges. Figure 3-1 shows a detention basin schematic with a small forebay at the inlet which also serves as a wetlands area. Figure 3-2 shows a detention basin schematic with extended release.

3.3 PROPOSED FUNCTIONS

Detention basins can be designed to either control stormwater runoff quantity, enhance stormwater runoff quality, or both. The location of the proposed basin will determine the minimum required performance standards. Table 3-1 describes the four primary basin location scenarios, and the associated detention basin functions existing within the Hampton Roads area.

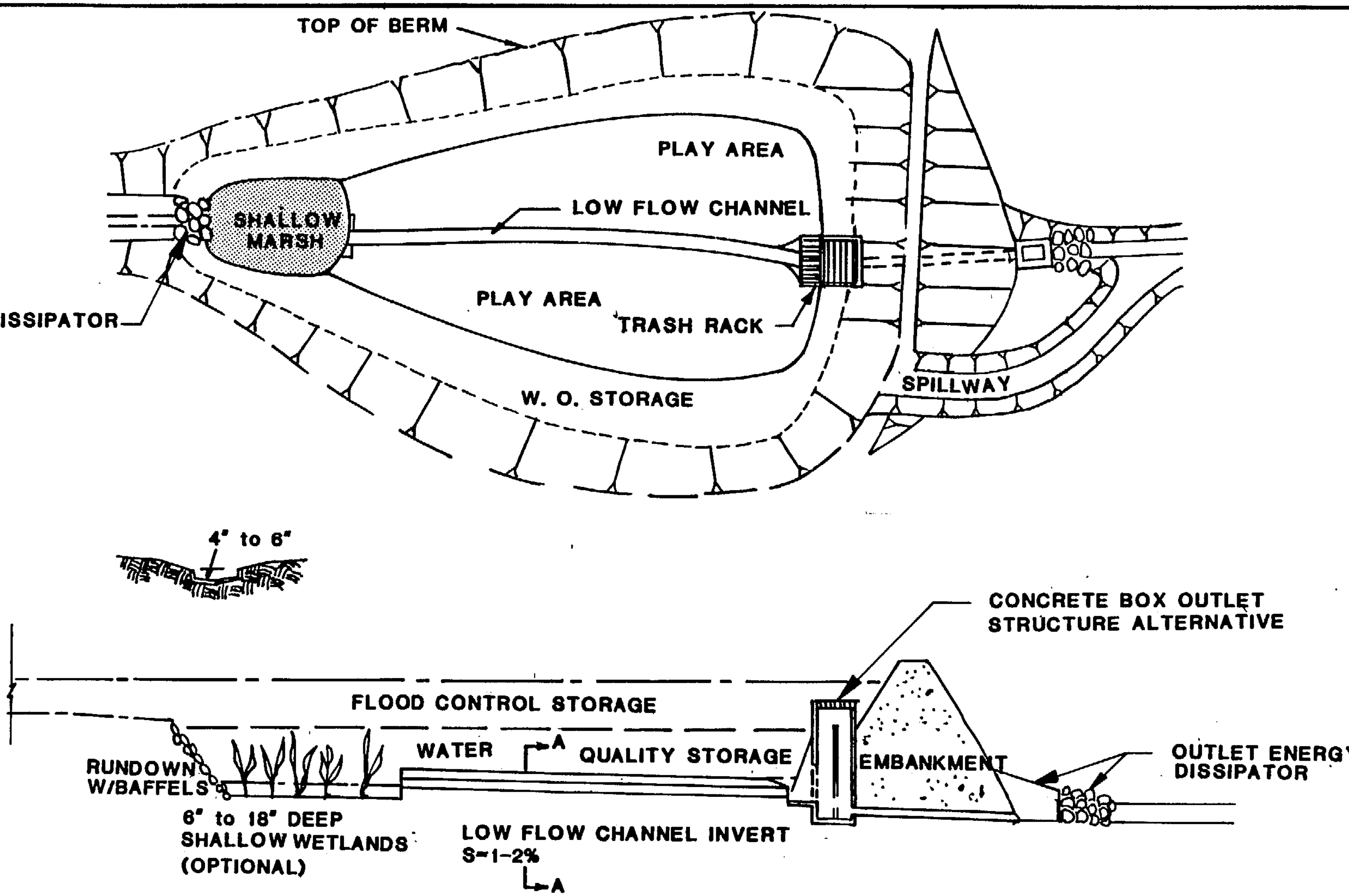

FIGURE 3-1
DETENTION BASIN SCHEMATIC

(Source- Stormwater Detention, Stahre, Urbonas, 1990)

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TABLE 3-1

LOCATION - DETENTION REQUIREMENTS

LOCATION	DETENTION REQUIREMENT
Locality with no CBPA or local stormwater management plan	None
Locality with local stormwater management plan	Quantity and quality controls under local guidelines (must be Commonwealth of Virginia requirements at a minimum)
Locality with CBPA requirements only	Quality controls under CBPA Guidelines
Locality with both CBPA and local stormwater management requirements	Quantity controls under local Guidelines. Quality controls under CBPA Guidelines

If both programs are in place, the more stringent water quality requirement - CBPA or local stormwater - will govern.

The design guidelines for each proposed basin function must be considered individually and then integrated if necessary for the overall final basin design. The following paragraphs describe the two primary basin functions, quantity and quality control, and how those functions must be addressed.

3.3.1 STORMWATER QUANTITY CONTROL

Temporarily storing or detaining excess stormwater runoff and then releasing it at a regulated rate has been a fundamental principle in stormwater management. The primary functions of a detention basin for stormwater quantity control is to reduce the post-development runoff from a development site or drainage area to a specified level, such as the pre-development rate. The state Stormwater Management Regulations require such a control strategy for at least the 2-year and 10-year design storms for basins constructed by State agencies and within localities that have implemented stormwater management programs. The regulations also include a water quality control component.

3.3.2 STORMWATER QUALITY ENHANCEMENT

The design of detention basins for stormwater quality enhancement is still a relatively new process. Collection of qualitative data supporting the design guidelines used to date has only recently begun. Water quality enhancement functions of detention basins are a primary consideration in both the state Stormwater Management Regulations and Chesapeake Bay Preservation Act Regulations; however, as shown in Table 3-1, any locality adopting requirements for detention basins must require some level of water quality enhancement. The

Stormwater Management Regulations require the first one-half inch of runoff from the total development area to be stored and released over a minimum of thirty hours from the time of peak storage of the one-half inch of runoff. The Chesapeake Bay guidance establishes a performance criterion.

Detention basins can provide pollutant removal by both physical and biochemical processes. Larger, suspended pollutants such as sediment and other solids are mostly removed by settling under relatively quiescent conditions. Extending the stormwater detention time to a period much longer than required for basic quantity control allows the necessary quiescent conditions to develop within the basin along with providing additional time for settling to occur. Smaller sizes pollutants and those which are more likely to be dissolved in the stormwater, such as nutrients, require biochemical activity for removal. The establishment of wetland areas, or shallow marsh areas in the basin bottom provides a region for that activity to occur. The determination of the primary pollutant of concern, as well as the proposed basin location, are the key considerations in the selection of basin features necessary to provide the required water quality enhancement function.

3.4 DESIGN GUIDELINES

This section contains the recommended design guidelines for detention basin BMPs. These guidelines are the result of compiling design data that is in use within the HRPDC area, design data from other areas of the state and country, the requirements of the Virginia Stormwater Management Regulations and the requirements of the Chesapeake Bay Preservation Act. The guidelines are intended to provide the general procedures necessary for designing detention basin BMPs to achieve the required quantity and quality control functions. The guidelines are not intended to stifle the innovative engineering processes necessary in basin design or to supersede local requirements.

As such, this manual does not provide step-by-step, "cookbook" design procedures or individual design examples. The guidelines are intended for use by a professional engineer experienced in drainage design and stormwater management who can apply the appropriate knowledge and insight to produce an effective and efficient basin design.

The following sections describe the methodology of the computations and the minimum standard physical features required for successful detention basin design for both quantity and quality control. Also discussed are design modifications and alternatives that can be implemented to provide different basin operational functions, if desired.

3.4.1 QUANTITY CONTROL GUIDELINES

This section contains design guidelines for detention basins to be used for quantity control. They should be utilized in conjunction with the quality control guidelines described in Section 3.4.2 where necessary to ensure the basin performs all of its required functions.

3.4.1.1 METHODOLOGY OF COMPUTATIONS

Stormwater quantity control in detention basins is primarily a function of watershed hydrology and basin hydraulics. The primary design computations necessary involve the inflow and discharge hydrographs and the outlet structure hydraulics. Basic to these calculations are the selection of an appropriate design storm and the basin outlet rate. The allowable outlet rate may be based on historic, or pre-development, runoff levels, or the discharge capacity of the downstream system. Placing limits on the volume of runoff allowable may also be considered.

<u>Design Storm</u>: The design storm is a primary component in basin design. The storm return period and duration should be chosen to both reflect the characteristics of the watershed and to meet local regulatory requirements. It is recommended that, unless local hydraulic conditions require other specific control standards, all detention basins be designed to control the 2-year and 10-year storms through the outlet structure, with an emergency outlet or spillway capable of passing the flows from the 100-year storm as discussed in Section 1.7.

The design storm duration, if not otherwise specified by the locality,should be greater than or equal to the drainage area time of concentration. It has been shown in other studies that the 6-hour design storm provides good representation of the watershed drainage characteristics and allows for proper downstream routing of stormwater runoff. If the watershed time of concentration is between 1 and 6 hours, a 6-hour design storm duration may be used for hydrologic calculations. If the drainage area time of concentration is greater than 6 hours, a 24-hour design storm should be used. The SCS method uses a 24-hour design storm. The purpose of using the design storm is to provide rainfall data to compute the volume of runoff and in turn to evaluate storage capacity.

<u>Hydrograph Calculation</u>: Each selected design storm must then be utilized to calculate the inflow and discharge hydrographs of the proposed detention basin. It is important to determine the entire hydrograph and not just the peak runoff rate since the detention basin must be capable of controlling both the rate and volume of runoff from a drainage area. In the following discussion, the calculation of hydrographs pertains to each design storm which requires control by the basin.

The pre-development and post-development hydrographs for the drainage area should be calculated using an appropriate method such as those described in Section 2.2.2 of this manual. The pre-development hydrograph calculations should be based on the assumption that the land area prior to development exhibits hydrologic conditions typical for that type area. The post-development hydrograph calculations should be based on available predictions of the ultimate development for the entire drainage area tributary to the basin. This is especially important when the basin is intended to serve as a regional facility, and may be developed over a period of time of several years.

The post-development hydrographs from the drainage area become the inflow hydrographs to the detention basin. Inlet facilities must be designed to accommodate the range of flows expected from all of the design storms. The peak runoff rates indicated by the pre-development hydrographs typically become the limiting basin discharge for each selected design storm. For example, if the peak pre-development flows for the 2-, 10-, and 100-year storms for a drainage area are 10, 50, and 200 cubic feet per second (cfs), respectively, the basin and outlet structure would be designed to release flow at or below those levels under post-development conditions for each design storm, if required by the locality and if practical.

However, the state Stormwater Management Regulations also state that a developer may have to reduce post-development outflow rates to levels less than the pre-development rate in order to prevent flooding or erosion downstream. Localities may only impose this type of additional requirement if a watershed study has been done.

Basin Outlet Design: In order to accommodate the above design storm and hydrograph requirements, the inflow hydrographs must be hydraulically routed through the basin and a multi-stage outlet structure must be evaluated and designed. Routing provides a defined estimation of the timing of the flows into and out of the basin, along with predicting the actual volume of water requiring storage at any time during the storm. The outlet structure can include weirs, orifices, pipes, or a combination of these and other flow controlling configurations to provide the level of quantity control required for the appropriate design storms. An example of such a multi-stage outlet structure is shown in Figure 3-3.

The inflow hydrographs can be hydraulically routed through the basin by a number of manual and computerized procedures. One of the manual processes widely utilized is the Storage/Indication method, also called the Modified Puls method. There are also a number of commercially available programs for personal computers that increase the speed of the calculations and allow for relatively quick alternatives analysis.

The primary data required for any of the above methods or programs is the basin depth-versus-storage information and an initial estimate of the outlet structure configuration. The depth/storage data can be developed based upon the proposed size and shape of the basin, computing the volume of water stored for each increment of basin depth. The depth/storage data is then combined with the proposed outlet structure to determine the timing (routing) of the stormwater inflow, storage, and outflow. This will be an iterative process involving multiple calculations, alternative outlet structure designs, and other variations until the correct quantity control functions are met. By plotting the inflow hydrograph and

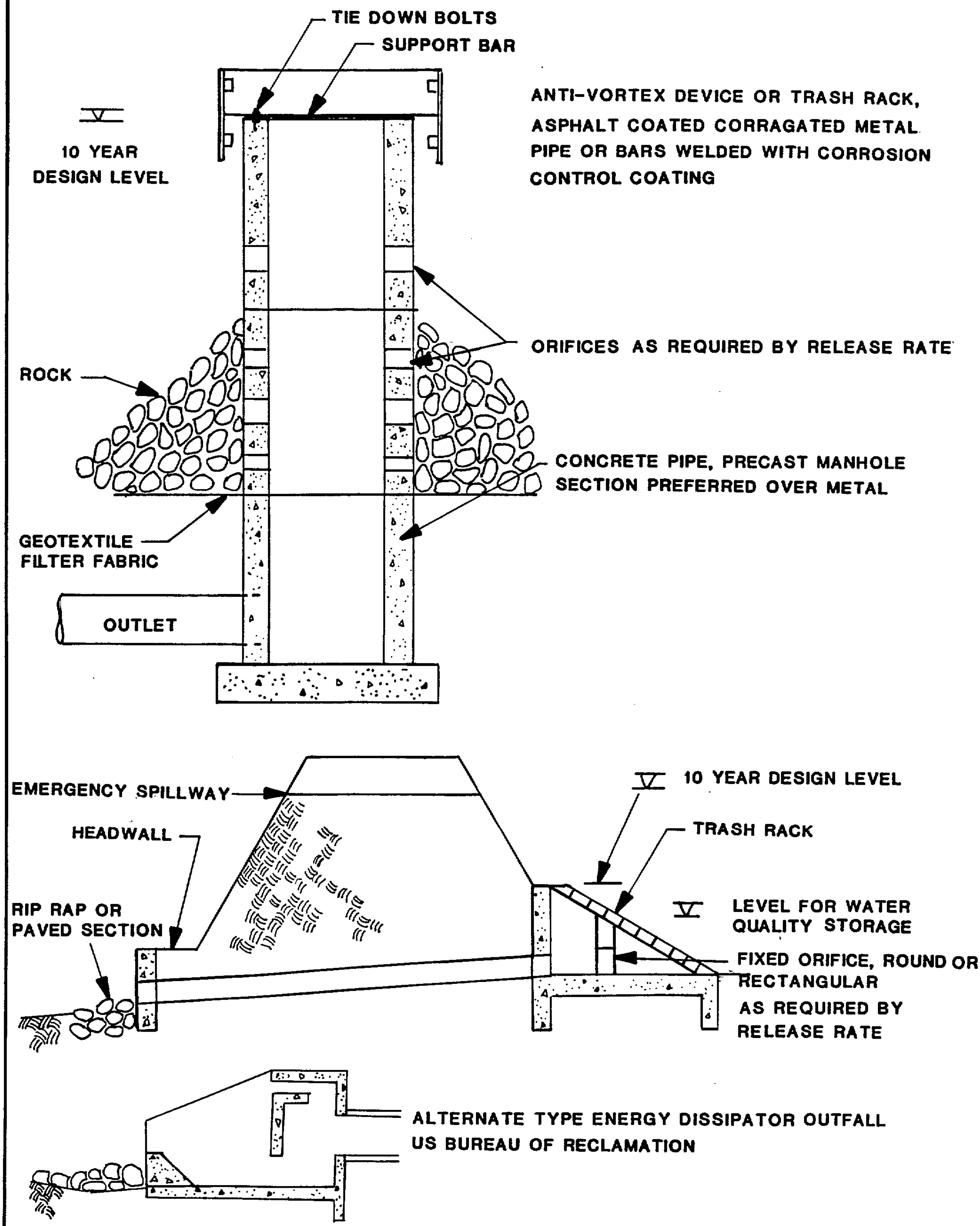

FIGURE 3-3
TYPICAL DETENTION BASIN OUTLET STRUCTURES FOR EXTENDED RELEASE

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outflow hydrograph as developed from the outfall structure discharge at various storage elevations, the volume of required storage can be determined.

3.4.1.2 PHYSICAL FEATURES OF A BASIC DETENTION BASIN

There are certain basic physical features of detention basins that have been found to increase the efficiency and effectiveness of the basin operation. The following design guidelines describe those features that can optimize the stormwater quantity control function and facilitate maintenance of the facility. These guidelines should be used as minimum requirements to produce satisfactory basin designs.

Side Slopes: The side slopes of the basin should be at a maximum slope of 3:1 horizontal to vertical for maintenance and ground cover control. If steeper slopes are required, they should be paved.

Low Flow Channel: A low flow channel should be provided through the basin to transport any dry weather flows and storm flows less than the minimum controlled design storm. The minimum slope through the basin for this channel should be 0.5 percent if paved and 2.0 percent if grass lined. In flat topography, a 2 percent slope may be difficult to obtain and the designer may have to modify the channel cross sections to optimize low flow velocities. The entire bottom should drain to stay dry.

Embankment: The basin height should allow for a minimum of 1 foot of freeboard above the elevation of maximum water storage. If the height of the embankment exceeds 25 feet from the downstream toe to the top, and the basin capacity is greater than 50 acre-feet, the Commonwealth of Virginia Dam Safety Regulations must be addressed.

Configuration: Oblong shapes are best with a minimum length to width ratio of 2:1.

Inflow structure: The design must consider protection against entrance erosion by paving or lowering the entrance channel so the flow enters the pool. The inflow needs to be distributed evenly into the pond to avoid stagnant zones and also to avoid short circuiting of the inflow directly to the outfall structure.

Outlet structure: Several types are possible. Safety needs to be considered. Trash racks should be installed, or a gravel or stone encasement be used.

3.4.2 QUALITY CONTROL GUIDELINES

This section contains design guidelines for use when detention basins are to achieve stormwater quality control as discussed in Section 2.3. They should be utilized in conjunction with the quantity control guidelines described in Section 3.4.1 where necessary to ensure the basin performs all of its required functions.

3.4.2.1 METHODOLOGY OF COMPUTATIONS

Stormwater quality control or enhancement in detention basins is primarily a function of the detention time available for solids and other pollutants to settle out of the flow. Sedimentation is the key process for removal of pollutants in detention basins. Alternative measures to provide for additional detention time and other potential treatment processes are described in Sections 3.4.3 and 5.0.

The requirements for a detention basin to provide water quality control will depend upon its location. As shown in Table 3-1, stormwater quality control or enhancement is required in areas under the jurisdiction of either a local stormwater management program developed under the Virginia Stormwater Management Regulations or the Chesapeake Bay Preservation Act. The minimum levels of stormwater quality control required, and the procedures for calculating and designing those levels, also depend upon the regulations to which the basin must conform.

Commonwealth of Virginia Stormwater Management Regulations: The Commonwealth of Virginia Stormwater Management Regulations (VR 215-02-00), while recommending planning on a regional or watershed basis, also impose some minimum restrictions on the enhancement of water quality through the use of detention basins. These requirements must be met in any locality adopting a stormwater management program in accordance with the Regulations.

The Virginia Regulations require that detention basins store a minimum "water quality volume" equal to the first 0.5-inch of runoff over the entire development area. The water quality volume must be released from the basin over a minimum 30-hour period to provide the desired retention and pollutant removal time. The remainder of any storage requirements in the basin will depend on the quantity control function. The primary quantity control issue addressed by the Regulations is the requirement that the post-development release from the basin not exceed the pre-development runoff from the development site for the 2-year and 10-year design storms, as nearly as practical.

Chesapeake Bay Preservation Act Requirements: The Chesapeake Bay Preservation Act establishes criteria relating to performance standards, best management practices, and planning and zoning concepts to protect the quality of state waters while allowing appropriate use and development of the land. While the standards do not directly address detention basins, the performance of any best management practice implemented within preservation areas designated under a local program must meet these requirements.

In general, the water quality enhancement goals of the CBPA include:

- For new development, the post-development nonpoint source pollution runoff load shall not exceed the pre-development load based upon average land cover conditions.

- The redevelopment of any site not currently served by water quality best management practices shall achieve at least a 10 percent reduction of nonpoint source pollution in runoff compared to the existing runoff load from the site. Post-development runoff from any site to be redeveloped that is currently served by water quality best management practices shall not exceed the existing load of nonpoint source pollution in surface runoff.

The CBLAD Local Assistance Manual includes a Guidance Calculation Procedure that outlines the steps needed to determine if a BMP meets the criteria. The Guidance Calculation Procedure is included in this manual as Appendix B. Because nonpoint source pollution can include many different contaminants and compounds, the calculation procedure is based upon the "keystone pollutant" concept. The keystone pollutant is an indicator pollutant, the existence of which provides an estimate of the total level of pollution in the runoff. The keystone pollutant for the Tidewater Virginia area is total phosphorus.

When a stormwater management facility is proposed outside of the CBPA (RPA or RMA), it is recommended that the projected water quality enhancement be calculated. Although other methods can be used, the Guidance Calculation Procedure provides an estimation. It has been recognized that the CBLAD procedure should not be used without understanding its limitations and lack of historical data. Long-term monitoring of all types of structural and non-structural BMPs will allow more detailed calculations of removal efficiencies for a variety of pollutants. It is important to evaluate each situation and not to apply blanket requirements arbitrarily. This is especially critical if the procedures or methods are used for regulatory or enforcement purposes.

3.4.2.2 PHYSICAL FEATURES FOR WATER QUALITY ENHANCEMENT

The physical design of detention basins for water quality enhancement is a relatively new procedure. There are, however, some features and configurations that have been shown to provide successful results to date. Since the minimum requirement for water quality enhancement in detention basins designed for programs adopted under the Virginia Stormwater Management Regulations is to release the "water quality volume" over a period of at least 30 hours, all such facilities will come under the heading of detention basins with extended release. The physical features requirements of these basins are described in Section 3.4.3.1.

3.4.3 DESIGN MODIFICATIONS FOR WATER QUALITY ENHANCEMENT

There are modifications that can be made to the basic design of detention basins to improve the removal of nonpoint source pollution. The selection of the appropriate modification for any particular site should be based on the type and degree of pollution removal desired. The CBPA Guidance Calculation Procedure includes steps for the proper selection of a BMP.

3.4.3.1 EXTENDED RELEASE

Extending the retention, or flow release, time of a detention basin is an effective means of implementing stormwater quality enhancement. Longer detention times allow for quiescent conditions to occur in the basin, facilitating settling and other pollutant removal processes. Extended release, used as water quality enhancement modification, is most effective for the removal of the larger, particulate pollutants. A release time of at least 24 hours has been shown to achieve as much as 90 percent removal of these materials.

As with other detention basins, there are certain design guidelines that can facilitate the performance of a basin with extended release. The following paragraphs describe some of the guidelines that have been found to be successful.

Release Times: The release time is the primary factor in the removal efficiency of an extended release basin. The Virginia Stormwater Management Regulations provide good minimum standards to evaluate during the basin design. The Regulations require a minimum release time for the "water quality volume" portion of the basin contents of 30 hours starting at the time of peak basin storage. This should result in an average detention time for all the flow of about 12 hours. Release times must be analyzed in conjunction with removal efficiencies that may be required if the basin is located in a CBPA regulated area.

Basin Configuration: The basin size and shape has a direct effect on the flow-through and settling characteristics of the stormwater. Oblong shaped basins are most effective. A minimum length to width ratio of 2:1 is recommended to help prevent short-circuiting of the flow through the basin. Side slopes should be no steeper than 3:1 horizontal to vertical for maintenance and slope protection.

Forebay: A forebay is a section of the inlet area of the basin designed to intercept the larger particles for settling and thus facilitate their eventual removal by keeping them out of the deeper portions of the basin. The forebay should include a baffle of wood or concrete or some other appropriate material on its downstream side to slow the flow, improve the particulate capture efficiency, and aid in the prevention of short-circuiting. The forebay is indicated in Figure 3-1. Since the forebay area acts like a sediment basin or trap, the area will need frequent maintenance. The area needs to be easily accessed by an all-weather roadway for heavy vehicles. Plantings or wetlands mitigation need to be avoided in this area. In small basins, this area could be paved for easy maintenance. The use of rip-rap in these areas should be avoided.

3.4.3.2 INFILTRATION BASINS

The design of the basic detention basin can also be modified so that it functions as an infiltration basin. An infiltration basin is a detention facility without a primary outlet structure so that the stormwater runoff infiltrates into the ground. It functions in a similar manner to a detention basin when the basin inflow exceeds the infiltration capacity, and water is stored until it can infiltrate. Infiltration basins can be effective in removing both soluble and fine particulate pollutants. Larger pollutants must typically be removed from the flow before it enters an infiltration basin. The overall infiltration basin design is similar to other detention facilities, with the goal being that the facility will contain the design inflow without overflowing. A schematic of an infiltration basin is shown in Figure 3-4.

The use of infiltration basins in the Tidewater Virginia region is restricted by state Stormwater Management Regulations to areas where the basin invert can be at least 4 feet above the local high groundwater level. While this effectively eliminates many areas in the Hampton Roads vicinity, infiltration basins remain, in concept, a viable, if limited, stormwater BMP for the Hampton Roads area. Guidelines for their basic design and configuration, therefore, have been included in this manual.

<u>Site Selection</u>: The soils and groundwater levels must be investigated specifically for a potential basin site. Each soil core must extend at least 5 feet below the proposed basin floor elevation. Soils within this zone should have a minimum field infiltration rate of 0.5 inches/hour as a desirable rate; however, rates as low as 0.25 inches per hour have been considered acceptable, especially for smaller facilities such as infiltration trenches. A table of infiltration rates for smaller facilties can be found in the Phase I companion manual to this document. The lowest measured infiltration rate, as measured by a percolation test, indicated at the site of the proposed basin should be used for the design calculations.

As described above, and as required by the state Stormwater Management Regulations, the basin floor must also be at least 4 feet above the seasonally high local groundwater level. Additionally, a site must not be used for an infiltration basin if any of the following conditions exist:

- bedrock is within 4 feet of the basin floor
- the site is over fill material
- the surface and underlying soils are classified in the SCS Hydrologic Soil Group "D".

As an example, Table 3-2 below lists some SCS soil groups, typical soils within each group, and a general average infiltration rate for each soil.

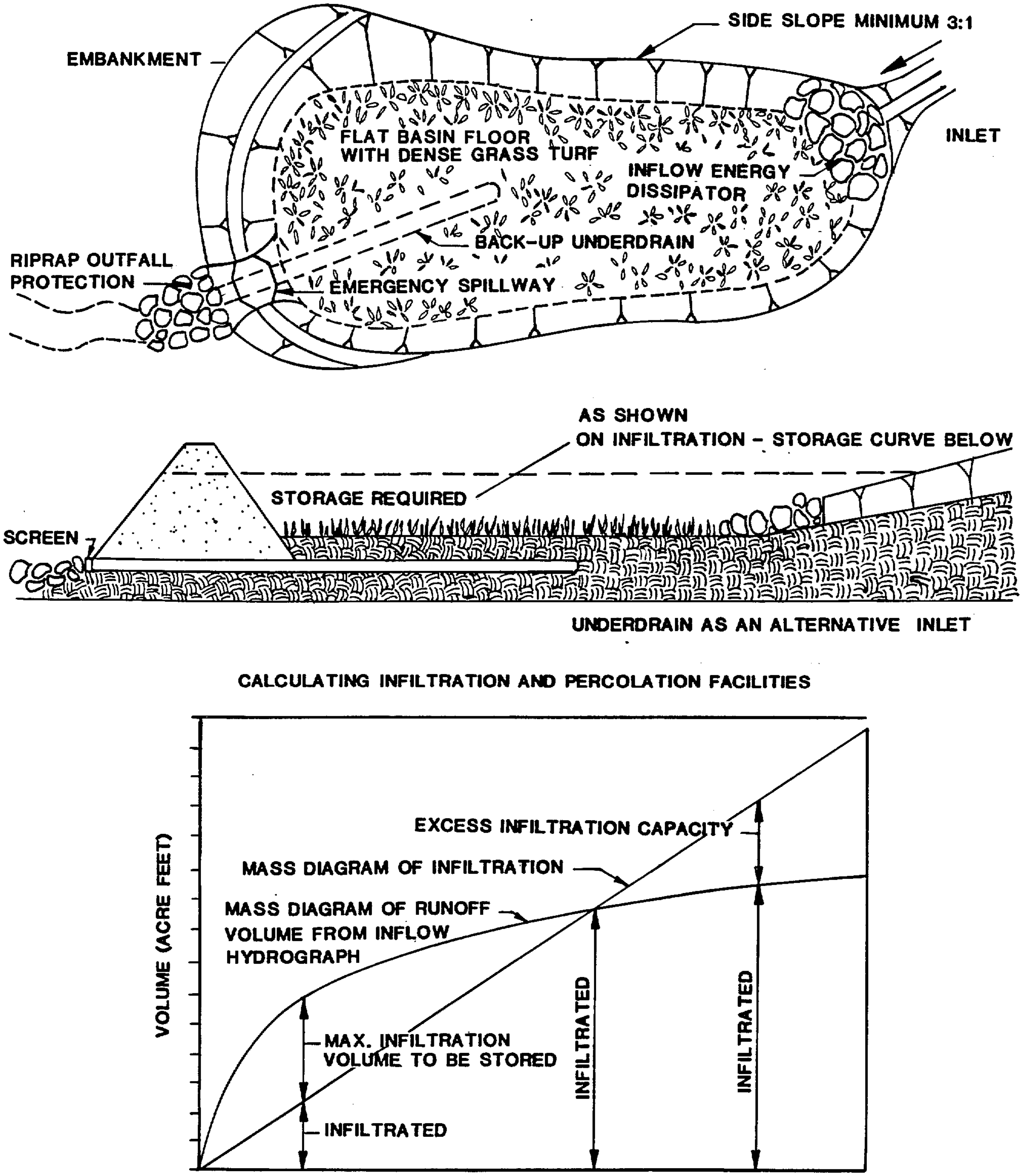

FIGURE 3-4
SCHEMATIC INFILTRATION BASIN

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TABLE 3-2
TYPICAL INFILTRATION RATES

SCS Soil Group and Soil	Infiltration Rate (in/hr)
A. Sand	8.0
A. Loamy Sand	2.0
B. Sandy Loam	1.0
B. Loam	0.5
C. Silt Loam	0.25
C. Sandy Clay Loam	0.15
D. Clay Loam and Silty Clay Loam	<0.09
D. Clays	<0.05

(Source: Stormwater Detention, Stahre and Urbonas, 1990.)

The basin should be capable of completely infiltrating the first 0.5 inch of runoff per impervious acre of contributing watershed. This minimum requirement can achieve significant pollutant removal and downstream channel protection.

Infiltration basins should only be used on drainage areas less than or equal to 25 acres in size.

Basin Configuration: The preliminary basin size can be estimated by the general rule that the infiltration surface area should not be smaller than one-half of the tributary impervious surface area. Final size needs to be calculated considering the infiltration rate. The basin size can also be estimated by plotting the runoff hydrograph and the estimated stormwater infiltration volume versus time. The greatest difference between the runoff and infiltration plots indicates the maximum amount of water that will have to be stored which is also shown on Figure 3-4.

In general, the storage depth should be adjusted so that the basin completely drains within 48 hours.

The side slopes, like detention basins, should have maximum slopes of 3:1 horizontal-to-vertical to allow for maintenance and bank stabilization.

The basin floor should be graded as flat as possible to permit uniform ponding and infiltration. Low spots and depressions should be leveled out.

Basin Inlets: The inlet pipe or channel leading to the basin should discharge at the same elevation as the basin floor.

All basins should also have sediment forebays or riprap aprons that dissipate the velocity of the incoming flow and trap larger sediments before they reach the basin floor.

3.4.3.3 RETROFITTING EXISTING FACILITIES

Many detention basins exist that do not have an outlet structure to allow for detention for periods of time approaching the thirty hours required to meet the extended release requirements for water quality enhancement. These basins often have side slopes that are steeper than 3:1 and bottoms that are flat, do not have a low flow or trickle flow channel, and are not graded to prevent pools of standing water. Most of these basins have an outlet structures that controls one flood, usually the five- or ten-year storm. The steps to retrofit should include:

1) Regrade slopes to 3:1 or less;

2) Regrade bottom to prevent standing water;

3) Install low flow channel;

4) Investigate inflow structures for erosion and repair;

5) Evaluate the outfall structure to incorporate the possibility of controlling a two-year storm or a second event to improve control over a wider variety of recurrence intervals;

6) Evaluate storage capacity for extended releases. Often the outfall structure is designed for pre-development flows from a 10-year storm and storage may be available for extended release of lower frequency storms even though it may not be possible for the design storm. If this is the case, the outlet structure can be modified to control a 2-year storm with extended storage and release;

7) Consider the construction of a forebay near the inflow for capturing heavier sediments. The forebay should be sized to store a volume equal to the annual runoff event for five minutes of detention. The bottom of the forebay should be stabilized for easy maintenance.

8) Consider adding a wetland bottom or a wetland finger along the low flow channel. This bottom area can be placed anywhere in the basin from the inlet to the outlet. It may be part of the forebay area if the forebay is not stabilized, and it is recognized that maintenance will disrupt the wetlands planting.

9) Consider increasing storage by adding height to the embankment.

3.4.3.4 WETLAND AREA ESTABLISHMENT

Establishing a wetlands area in a detention basin involves creating a shallow marsh located within the basin bottom or along the low flow channel. The low flow channel can meander through the basin with a marsh fringe on either side. Virginia does not have any guidelines for a shallow permanent pool in a detention basin designed for a wetlands area. The wetlands pool should be considered as a permanent pool in a retention basin where the volume is governed by the factor of three (3) times the water quality runoff. The wetlands pool has a different function and needs to be considered for its own benefits. Further details on design methods can be found in Section 5 of this manual.

3.5 CONSTRUCTION AND OPERATION ISSUES

The construction, operation, and maintenance of all types of detention facilities are primary factors in their success rate and longevity. A basin can be designed utilizing proven criteria and state-of-the-art techniques, but unless it is constructed according to that design and maintained so that it continues to emulate the original design, it will not be able to operate efficiently and achieve its desired water quantity and quality control functions.

The following sections contain guidelines for successful construction, operation, and maintenance of detention basin BMPs. It is incumbent on the administering locality that these guidelines, along with other proven and accepted techniques, be adhered to throughout the operating life of the facility.

3.5.1 CONSTRUCTION GUIDELINES

Construction Sequencing: The detention basin is typically part of the site plan as well as the erosion control program. Since it is designed to trap sediment from upstream development, the detention basin should be constructed in the early stages of the project. Grading operations should be scheduled in a manner which will limit the soil's exposure to erosion. Grade only those areas ready for immediate development. Promptly reseed bare soil upon completion of sitework. Additionally, protect downstream areas by installing graveled construction entrances, silt fence, check dams and other temporary measures until the facility is completed. Inspect the basin after it has been stabilized for premature silting, channel scour and other defects.

Site Layout and Preparation: The outside perimeter of the detention facility should be staked out before any clearing and grading begins. The embankment and any appurtenant work like stream bank stabilization should also be staked at this time. At a minimum, the following layout stakes, marked for grade, should be used:

- top of slope of the basin excavation
- bottom of slope of the basin excavation

- centerline of embankment
- front and back toe of slope or embankment
- several grade stakes through the basin floor

The first stakes to be set should be the centerline of the embankment and the top of the slope of the basin excavation.

The outlet control structure should be staked, constructed, and backfilled before general earthmoving is started.

The site must be dry for successful excavation to take place. If a site is wet, or if the site is expected to be wet during construction, measures should be taken to ensure proper conditions. These measures could include direct drainage trenches to points of lower elevation or the collection of runoff and surface water in sumps that require pumping.

Embankment Construction: Good fill material, suitable soils, and proper compaction techniques are imperative for the construction of a stable embankment. Increasing the embankment breadth and decreasing the slope can also be important measures.

Placing embankment fill should be performed in sequential lifts of 6- to 8-inches each. An entire lift across the embankment should be completed before the next lift is begun. This allows any moist soils to dry and additional compaction to occur from the application equipment.

The proper construction of a cutoff trench is imperative to prevent any undermining of the embankment. A cutoff trench is a trench excavated along the centerline of the embankment before the fill materials are placed. It must be constructed from a relatively impermeable soil. The cutoff trench can be constructed wide enough for the bulldozer or other equipment to work within it. The impermeable soils should be placed in 6- to 8-inch lifts. The cutoff trench must extend from several feet below the existing grade up into the embankment fill.

The placement of antiseep collars at the point where the outlet pipe passes through the embankment to prevent soil piping failures is of key importance. An antiseep collar is a metal, concrete, or masonry shield placed around the pipe within the fill embankment. The backfill material around the outflow pipe should also be properly placed and compacted to help prevent embankment failure.

Inflow and Outflow Structures: The inflow structure is generally less critical than the outflow structure, but it still requires accurate vertical placement. Slope protection should be used ahead of a stream inflow structure, downstream of an inflow "spillway" of any length, and at the basin discharge point. The inflow control structure must be constructed so that it directs the flow into the basin forebay, or across the basin floor as intended by the basin design. Riprap should be grouted to make future maintenance operations easier.

The outflow structure may contain several key components that must work in concert with each other. These may include weirs, orifices, grates, or other flow control sections. These must all be properly constructed and placed at accurate elevations. If the structure is constructed offsite, it must be inspected carefully upon delivery to the site for any defects or misalignments of any of the components. The final placement and/or construction must be exactly as shown on the construction documents.

Construction Operations: A retention basin is most subject to externally caused damage during its construction. Since most basins will be located at the low points of a site, they must be protected from extreme rainfall events that may occur during construction. Vegetative cover and the emergency spillway must also be completed as quickly as possible during the construction phase.

The use of an inspector is one of the best methods of ensuring that the detention basin is constructed as designed. This inspector may be an in-house representative, someone from the designing firm, or from an outside consultant or inspection company. The inspector may be full time or part time. The primary focus of the inspections should include:

- embankment fill placement
- embankment fill material
- implementation of adequate erosion and sediment control

Additional details can be found in Section 6 of this document.

3.5.1.1 INFILTRATION BASIN CONSTRUCTION GUIDELINES

Proper construction techniques are extremely important to the successful installation and operation of infiltration basins. The most common cause of infiltration basin failure is premature loss of infiltration capacity, which is often linked to the procedures followed during construction. The following guidelines, along with those described above, must be addressed during the construction of an infiltration basin.

- Heavy equipment traffic must be restricted from the basin area to prevent excessive soil compaction.
- If the basin is not intended to function as a sediment basin during construction, proper erosion and sediment control measures must be implemented prior to initiating the basin construction to keep excessive sediment from entering the basin.
- If the basin is to be used as a sediment basin during construction, initial grading should be completed to within only two feet of the final basin floor elevation. The final two feet can then be removed with the collected

sediment when the site is completely stabilized.

- The basin should be excavated using light earth-moving equipment with tracks or over-sized tires. Since some compaction will still occur, the basin floor should be tilled with a rotary tiller or disc harrow. The floor can then be smoothed and leveled after the excavation is complete.
- Slope stabilization with vegetation should be completed as soon as possible after construction.

3.5.2 COST ESTIMATES

The graph in Figure 3-5 shows the average cost to construct a detention basin of a size which would be required for a given total project area. Likewise, the graph in Figure 3-6 compares the construction cost associated with the volume of stormwater which must be detained. These figures are generic and should only be used as guidelines and planning purposes.

The costs shown in Figure 3-5 and 3-6 are based on generalized development scenarios which were also used in developing Figure 2-1. The basins were designed for a 10-year runoff. The retention basin includes water quality storage. The cost includes site preparation, earthwork, inlet, outlet structures, discharge energy dissipators, seeding, and a contingency for other costs.

The pre-development C-factors used for the associated hydrographs ranged from 0.35 for a small 1-acre site to 0.20 for a larger 50-acre site. Post-development C-factors were determined by the types of development typically associated with various sizes of land. These post-development C-factors ranged from 0.80 for small sites to 0.50 for larger sites. The costs of constructing detention facilities were generated from recent contractor estimates of various basin sizes and were compared with Means Sitework and Landscape Cost Data, 1991.

3.5.3 FACILITY LIFE EXPECTANCY

The life expectancy of a detention facility is directly proportional to the quality of construction and maintenance. If properly placed in stable soil, concrete can be expected to last up to fifty-years. The embankment properly constructed would have an indefinite life probably exceeding all other parts of the facility. Metal portions can be expected to last twenty-years or more if properly maintained. Aluminum alloy products will have a longer life if properly specified. Preventative and corrective maintenance are crucial to the success of the forebay and the basin bottom.

3.5.4 MAINTENANCE REQUIREMENTS

The agency responsible for long term maintenance must be identified during the planning stages. Even though a detention facility only performs its design role on an occasional basis, it must be constantly prepared to do so. A comprehensive,

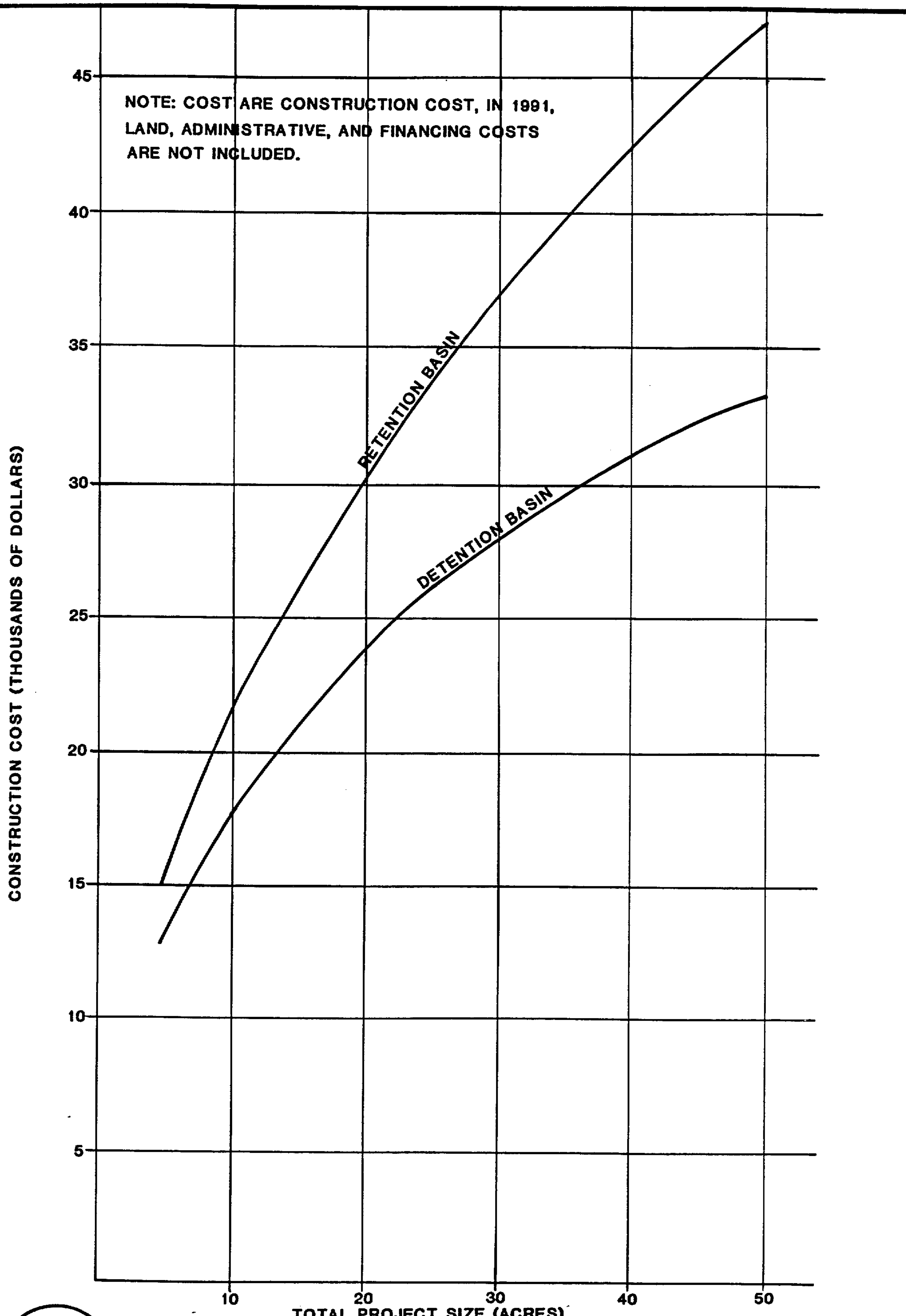

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FIGURE 3-5
TOTAL PROJECT SIZE VS. FACILITY COST

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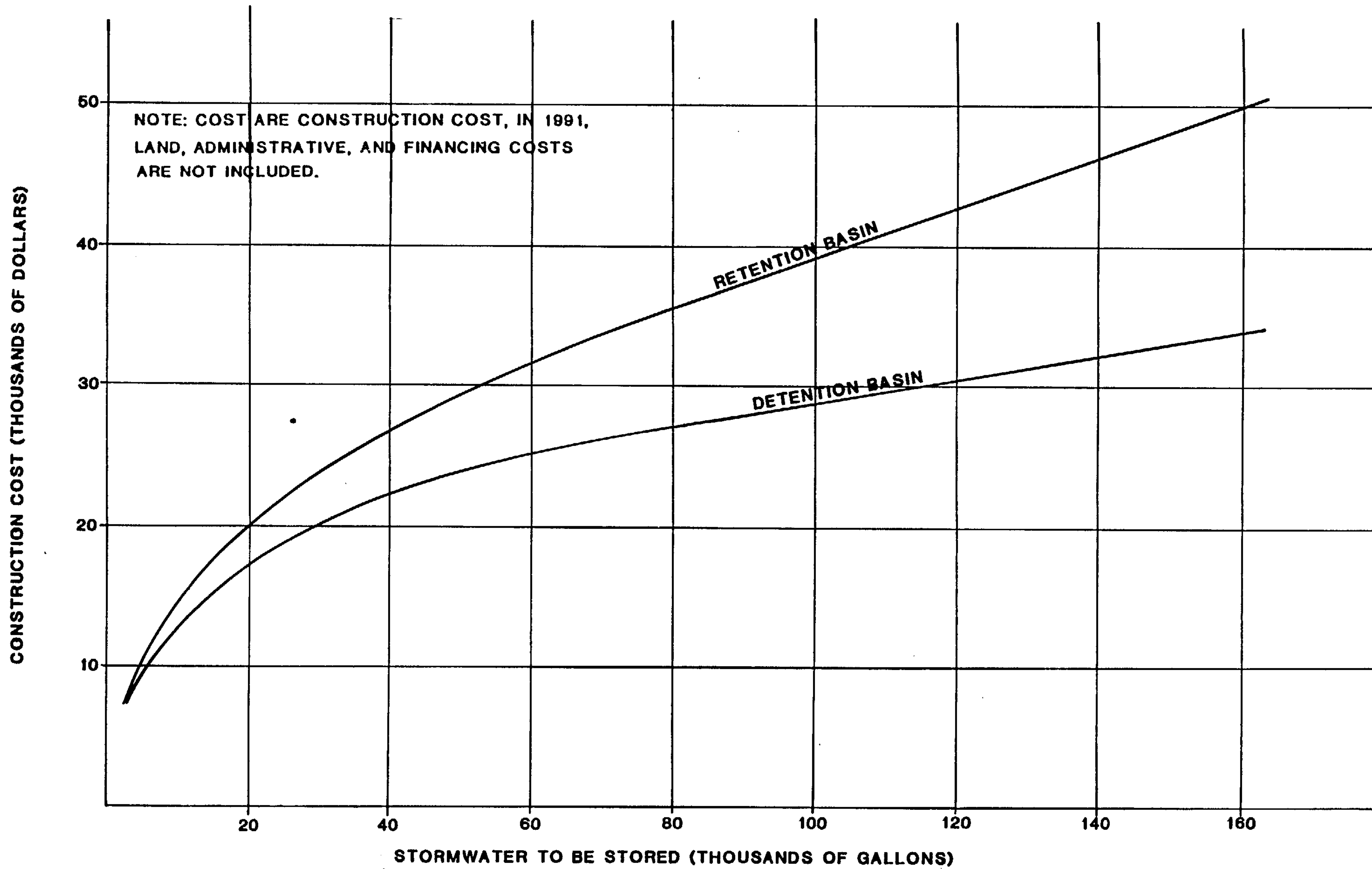

FIGURE 3-6

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CONSTRUCTION COST VS. STORMWATER TO BE STORED

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regularly-scheduled maintenance program is the key to any successful stormwater management facility. Such a program is comprised of funding, maintenance, inspection, training and program reviews.

A detention basin will be useless if funding for its maintenance is not adequate. Funding considerations include: staffing, equipment, and material needs; facilities for storage of materials; storage, maintenance and replacement of equipment; training and administrative costs; seasonal effects; long-term capital improvements; and emergency appropriations for unforeseen problems.

The physical portion of the maintenance program should include aesthetic, preventive and corrective measures.

Regular, major and informal inspections should be performed. Major inspections should be performed semi-annually and after each major storm. Regular inspections should be conducted to determine the need for and the effectiveness of maintenance work. Informal inspections should be conducted during every visit to the facility by maintenance personnel, and, if possible, prior to the occurrence of a major storm. Section 6 of this manual provides additional details on inspections.

A training program should include: maintenance and inspection techniques, proper record keeping,and stormwater program goals and objectives. Particular attention should be paid to the purpose and operation of stormwater management facilities, the importance of thorough maintenance, and the health, safety and other consequences of maintenance neglect.

Additional information can be found in Section 6 of this document.

3.6 PLAN SUBMITTAL REQUIREMENTS

Submittals for detention facilities need to be made to the locality in accordance with local ordinances and regulations for Stormwater Management, Erosion and Sediment Control, and Chesapeake Bay Preservation Ordinances. Only State agencies need to submit plans to the Division of Soil and Water Conservation and the Chesapeake Bay Local Assistance Department.

3.6.1 AGENCIES

The Department of Conservation and Recreation Division of Soil and Water Conservation can provide assistance on the Erosion and Sediment Control Law, Stormwater Management Act, and Dam Safety Act and Regulations.

The State Water Control Board, Permits Section can provide information on the stormwater NPDES permit.

The Chesapeake Bay Local Assistance Department can provide information on the Chesapeake Bay Preservation Act and Regulations.

The Norfolk District Corps of Engineers Construction Operations, Regulatory Permit Section, issues permits for wetlands disturbance and navigable water crossings.

VMRC and local Wetlands Boards need to be contacted for wetlands or projects impacting the shoreline.

If a permit is needed for construction because of wetlands disturbance or interference with a navigable stream, then a permit would probably be needed for maintenance dredging. If no permit is issued or needed for construction, then no permit would probably be needed for maintenance dredging. Dredge materials under either circumstance would probably not be regulated. At this point in time, there are no requirements for disposing of materials dredged from retention or detention basins; however, it would be prudent to discuss the issue with the Corps of Engineers prior to dredging.

3.6.2 SUBMITTAL CHECKLIST

A checklist has been prepared for overall guidance and can be found in Appendix A. Localities may have their own checklist which the applicant would need to follow. In addition, the Division of Soil and Water Conservation has developed a checklist to be used by State Agencies.

4.0 RETENTION BASINS

4.1 DESCRIPTION

A retention basin is a stormwater management facility comprised of: a) a permanent pool of water to enhance water quality which loses water primarily through infiltration and evaporation; and b) additional capacity above the permanent pool for the storage of stormwater runoff. The facility discharges to the downstream conveyance system through an outlet structure designed to both release the runoff over a specified period of time and maintain a minimum level of the permanent pool. These facilities are also called wet ponds or wet detention basins and can be used for both stormwater quantity and quality control.

4.2 APPLICABILITY

The use of retention basins as BMPs is most applicable in residential or commercial developments where there is a reliable source of water to maintain the permanent pool. Like detention basins, they can serve the dual function of stormwater quantity and quality control. They also can be an aesthetic, attractive feature in a development if designed. constructed, operated, and maintained correctly.

4.3 PROPOSED FUNCTIONS

Retention basins can be designed to either control stormwater runoff quantity, enhance stormwater runoff quality, or both. The location of the proposed basin will determine the minimum required performance standards. Table 4-1 describes the four primary basin location scenarios and the associated retention basin functions existing within the Hampton Roads area.

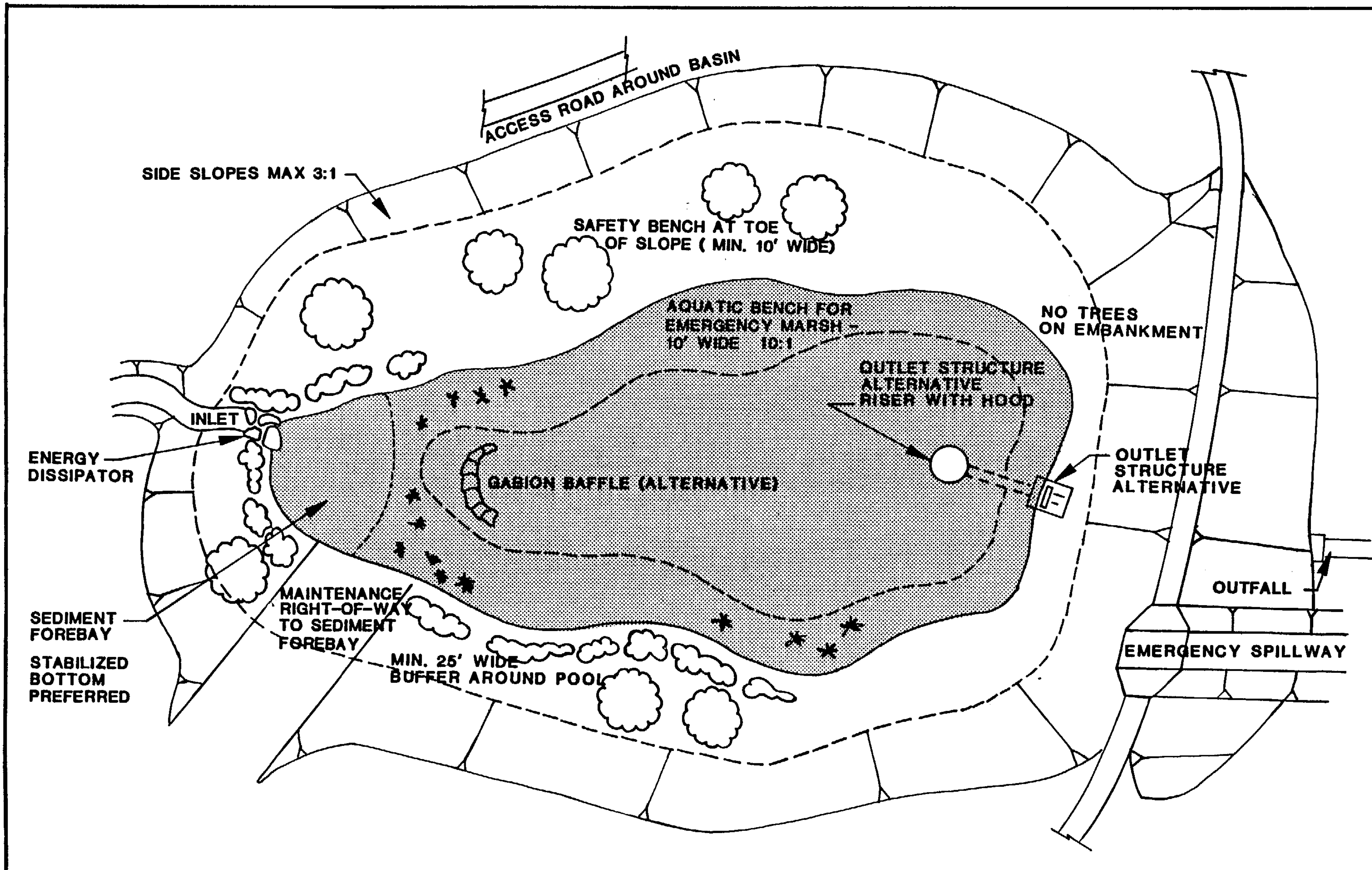

FIGURE 4-1
RETENTION BASIN SCHEMATIC

TABLE 4 -1

LOCATION - RETENTION REQUIREMENTS

LOCATION	RETENTION REQUIREMENT
Locality with no CBPA or local stormwater management plan	None
Locality with local stormwater management plan	Quantity and quality controls under local guidelines (must be Commonwealth of Virginia requirements at a minimum)
Locality with CBPA requirements only	Quality controls under CBPA Guidelines
Locality with both CBPA and local stormwater management requirements	Quantity controls under local Guidelines. Quality controls under CBPA Guidelines

If both programs are in place, the more stringent water quality requirement - CBPA or local stormwater - will govern.

The design guidelines for each proposed basin function must be considered individually and then integrated if necessary for the overall final basin design. The following paragraphs describe the two primary basin functions, quantity and quality control, and how those functions must be addressed.

4.3.1 STORMWATER QUANTITY CONTROL

Temporarily storing or detaining excess stormwater runoff and then releasing it at a regulated rate has been a fundamental principle in stormwater management. The primary functions of a Retention basin for stormwater quantity control is to reduce the post-development runoff from a development site or drainage area to or below pre-development levels. The state Stormwater Management Regulations require such a control strategy for at least the 2-year and 10-year design storms for basins constructed by State agencies and within localities that have implemented stormwater management programs. The regulations also include a water quality control component.

4.3.2 STORMWATER QUALITY ENHANCEMENT

Retention basins effect removal of pollutants from stormwater by two primary mechanisms, sedimentation (settling) and biological uptake. The relatively long detention times allow for the settlement of many of the larger suspended pollutants and the permanent pool and perimeter promote the conditions necessary for the biological activity to take place.

Theoretically, the inflowing stormwater displaces water out of the pond which is then stored until the next storm. The suspended pollutants settle out of the flow with the permanent pool acting to prevent their resuspension. The larger and coarser particles settle first, with the smaller and finer materials taking longer. Factors which can reduce the effectiveness of settling in retention basins are short-circuiting of the flow through the basin and an inflow volume that is greater than the permanent pool volume.

Aquatic plants and algae that can live in retention basins can remove significant quantities of soluble nutrients. They convert the nutrients into a mass that will settle into the sediments at the bottom of the pond.

4.4 DESIGN GUIDELINES

The following sections contain guidelines for the design of retention basin BMPs. Since retention basins can perform both quantity and quality control functions, guidelines are presented that specifically relate to each issue. The quantity and quality control functions must be evaluated separately and integrated into the final basin design.

This manual does not provide step-by-step, "cookbook" design procedures or individual design examples. The guidelines are intended for use by a professional engineer experienced in drainage design and stormwater management who can apply the appropriate knowledge and insight to produce an effective and efficient basin design. A schematic of a retention facility is shown on Figure 4-1.

The following sections describe the methodology of the computations and the minimum standard physical features required for successful retention basin design for both quantity and quality control. Also discussed are design modifications and alternatives that can be implemented to provide different basin operational functions, if desired.

4.4.1 QUANTITY CONTROL GUIDELINES

This section contains design guidelines for retention basins to be used for quantity control. They should be utilized in conjunction with the quality control guidelines described in Section 4.4.2 where necessary to ensure the basin performs all of its required functions.

4.4.1.1 METHODOLOGY OF COMPUTATIONS

Stormwater quantity control in retention basins, while somewhat more complex than in detention basins, is still primarily a function of watershed hydrology and basin hydraulics. The primary design computations necessarily involve the inflow and discharge hydrographs and the outlet structure hydraulics. Inherent in these calculations is the selection of an appropriate design storm or storms and the decision of how to limit basin outlet rates. The allowable outlet rates may be based on historic, or pre-development, runoff levels, the discharge capacity of the downstream system, or some specific value. Placing limits on the volume of runoff allowable may also be considered.

Design Storm: The design storm is a primary component in basin design. The storm return period and duration should be chosen to both reflect the characteristics of the watershed and to meet local regulatory requirements. It is recommended that, unless local hydraulic conditions require other specific control standards, all detention basins be designed to control the 2-year and 10-year storms through the outlet structure, with an emergency outlet or spillway capable of passing the flows from the 100-year storm as discussed in Section 1.7.

The design storm duration, if not otherwise specified by the locality, should be greater than or equal to the drainage area time of concentration. It has been shown in other studies that the 6-hour design storm provides good representation of the watershed drainage characteristics and allows for proper downstream routing of stormwater runoff. If the watershed time of concentration is between 1 and 6 hours, a 6-hour design storm duration may be used for hydrologic calculations. If the drainage area time of concentration is greater than 6 hours, a 24-hour design storm should be used. The SCS method uses the 24-hour design storm. The purpose of using the design storm is to provide rainfall data to compute the volume of runoff and in turn to evaluate storage capacity.

Hydrograph Calculation: Each selected design storm must then be utilized to calculate the inflow and discharge hydrographs of the stormwater storage portions of the proposed retention basin. It is important to determine the entire hydrograph and not just the peak runoff rate, since the retention basin must be capable of controlling both the rate and volume of runoff from a drainage area while maintaining the permanent pool level during non-storm conditions. In the following discussion, the calculation of hydrographs pertains to each design storm which requires control by the basin.

The pre-development and post-development hydrographs for the drainage area should be calculated using one of the appropriate methods described in Section 2.2.2 of this manual. The pre-development hydrograph calculations should be based on the assumption that the land area prior to development exhibits hydrologic conditions typical for that type of area. The post-development hydrograph calculations should be based on available predictions of the ultimate development for the entire drainage area tributary to the basin. This is especially important when the basin is intended to serve as a regional facility and may be developed over a period of several years.

The post-development hydrographs from the drainage area become the inflow hydrographs to the retention basin. Inlet facilities must be designed to accommodate the range of flows expected from all of the design storms. The peak runoff rates indicated by the pre-development hydrographs typically become the limiting basin discharge for each selected design storm. For example, if the peak pre-development flows for the 2-, 10-, and 100-year storms for a drainage area are 10, 50, and 200 cubic feet per second (cfs), respectively, the basin and stormwater control outlet structure would be designed to release flow at these or other approved levels under post-development conditions for each design storm if required by the locality and if practical.

However, the state Stormwater Management Regulations also state that a developer may have to reduce post-development outflow rates to levels less than the pre-development rate in order to prevent flooding or erosion downstream. Localities may only impose this type of additional requirement if a watershed study has been done.

<u>Basin Outlet Design</u>: In order to accommodate the above design storm and hydrograph requirements, the inflow hydrographs must be hydraulically routed through the basin and a multi-stage outlet structure must be evaluated and designed. This will be somewhat different than the design of a detention basin outlet structure since the "low flow" or smallest storm release level will still be above the permanent pool level. Routing provides a defined estimation of the timing of the flows into and out of the basin, along with predicting the actual volume of stormwater requiring storage at any time during the storm. The outlet structure can include weirs, orifices, pipes, or a combination of these and other flow controlling configurations to provide the level of quantity control required for the appropriate design storms. An example of such a multi-stage outlet structure is shown in Figure 4-2.

The inflow hydrographs can be hydraulically routed through the basin by a number of manual and computerized procedures. One of the manual processes widely utilized is the Storage/Indication method, also called the Modified Puls method. There are also a number of commercially available programs for personal computers that increase the speed of the calculations and allow for relatively quick alternatives analysis. It must be remembered that the method used must be able to accurately simulate a reservoir, since there will always be water already in the retention basin when the stormwater flows enter it.

The primary data required for any of the above methods or programs is the basin depth-versus-storage information and an initial estimate of the outlet structure configuration. The depth/storage data can be developed based upon the proposed size and shape of the basin, computing the volume of stormwater stored for each increment of basin depth above the permanent pool. The depth/storage data is then combined with the proposed outlet structure to determine the timing (routing) of the stormwater inflow, storage, and outflow. This will be an iterative process involving multiple calculations, alternative outlet structure designs, and other variations until the correct quantity control functions are met.

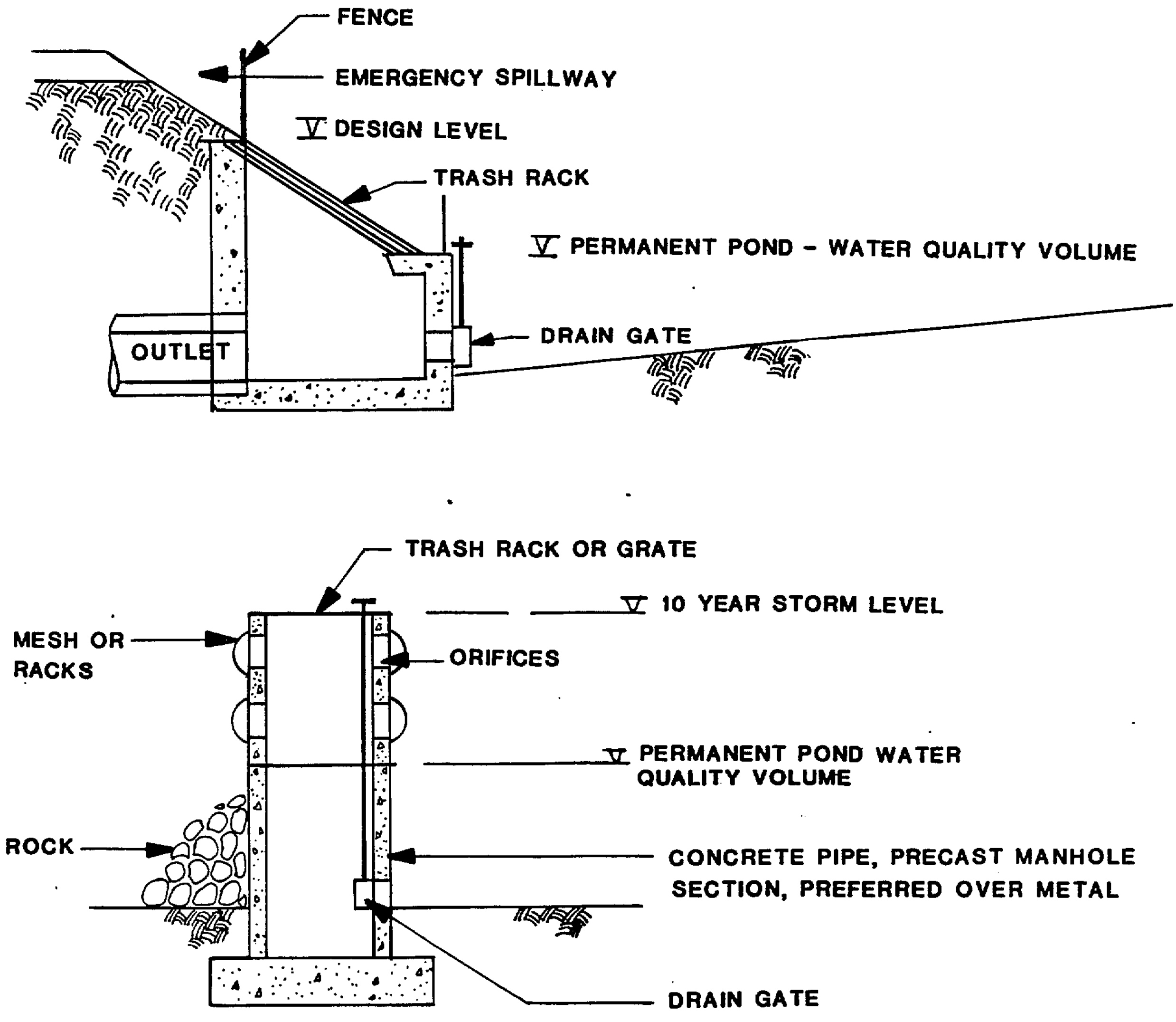

FIGURE 4-2
TYPICAL RETENTION BASIN
OUTLET STRUCTURES WITH PERMANENT POOL

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4.4.1.2 PHYSICAL FEATURES OF A BASIC RETENTION BASIN

There are certain basic physical features of retention basins that have been found to increase the efficiency and effectiveness of the basin operation. The following design guidelines describe those features that can optimize the stormwater quantity control function and facilitate maintenance of the facility. These guidelines should be used as minimum requirements to produce satisfactory basin designs.

Basin Side Slopes: The side slopes of the basin should be at a maximum slope of 3:1 horizontal to vertical for maintenance and ground cover control. It steeper slopes are required they should be paved.

Permanent Pool Side Slopes: The side slopes entering the water should taper off at a slope of 5:1 or less for the first ten feet where the slope can be increased to 3:1. This is to reduce the possibility of people or animals not being able to get out of the water should they fall in.

Embankment: The basin height should allow for a minimum of 1 foot of freeboard above the elevation of maximum water storage. If the height of the embankment exceeds 25 feet from the downstream toe to the top, and the basin capacity is greater than 50 acre-feet, the Commonwealth of Virginia Dam Safety Regulations must be addressed.

Pond Configuration: The pond should be configured to minimize short-circuiting of the stormwater flow. The most direct way to achieve this is to maximize the distance between the inlet and the outlet. A minimum length to width ratio of 3:1 is recommended. If the local site conditions inhibit the construction of a relatively long narrow facility, baffles or gabions or other materials should be placed within the pond to "lengthen" the stormwater flow path as much as possible.

The invert elevation of all inlet pipes should be at or within one foot below the surface of the permanent pool.

The low flow stormwater outlet orifice should be negatively sloped so that it draws water from at least one foot below the permanent pool surface.

Reinforced concrete should be used in the construction of all risers, barrels, and pipes in the stormwater outlet structure to provide for greater longevity. Corrugated metal should not be used.

The riser should be located within or at the face of the embankment where possible. This facilitates future maintenance and prevents flotation problems.

All retention basins should have a maintenance drain to allow the basin to be completely emptied for maintenance and repairs and sediment removal. This should be a gate valve or slide gate with a positive seating head.

Maintenance access must be provided. The access way should be a minimum of 10 feet wide, with widths of 15' being common, have a maximum slope of 5:1 (H:V), and never cross the emergency spillway. Slopes less than 5:1 are preferred and 10:1 are not uncommon. The steeper slopes will result in more rutting and access road maintenance.

Additional volume should be provided for sediment accumulation. A rule of thumb is to add 25% of the volume, but detail calculations based on watershed sediment yields and basin trap efficiency will provide a more accurate volume.

A sediment forebay should be constructed near the inlet to trap sediments entering the basin with the stormwater. Methods to calculate a required forebay volume can be found in the references, but in general the forebay should be able to detain the seasonal average inflow for about five minutes.

4.4.2 QUALITY CONTROL GUIDELINES

This section contains design guidelines for use when retention basins are to achieve stormwater quality control. They should be utilized in conjunction with the quantity control guidelines described in Section 4.4.1 where necessary to ensure the basin performs all of its required functions.

4.4.2.1 METHODOLOGY OF COMPUTATIONS

Stormwater quality control or enhancement in retention basins is a function of the detention time available for solids and other pollutants to settle out of the flow and the biological action that can take place in the permanent pool. Alternative measures to provide for additional detention time and other potential treatment processes are described in Sections 4.4.3 and 5.0.

The requirements for a retention basin to provide water quality control will depend upon its location. As shown in Table 4-1, stormwater quality control or enhancement is required in areas under the jurisdiction of either a local stormwater management program developed under the Virginia Stormwater Management Regulations or the Chesapeake Bay Preservation Act. The minimum levels of stormwater quality control required, and the procedures for calculating and designing those levels, also depend upon the regulations to which the basin must conform.

<u>State Stormwater Management Regulations</u>: The state Stormwater Management Regulations (VR 215-02-00), while recommending planning on a regional or watershed basis, also impose some minimum restrictions on the enhancement of water quality through the use of retention basins. These requirements must be met in any locality adopting a stormwater management program in accordance with the Regulations.

The Virginia Regulations require that the permanent pool volume in retention basins is at least 3 times the "water quality volume" (the first 0.5-inch of runoff over the entire development area). The remainder of any storage requirements in the basin will depend on the quantity control function.

Chesapeake Bay Preservation Act Requirements: The Chesapeake Bay Preservation Act establishes criteria relating to performance standards, best management practices, and planning and zoning concepts to protect the quality of state waters while allowing appropriate use and development of the land. While the standards do not directly address retention basins, the performance of any best management practice implemented within preservation areas designated under a local program must meet their requirements.

In general, the water quality enhancement goals of the CBPA include:

- For new development, the post-development nonpoint source pollution runoff load shall not exceed the pre-development load based upon average land cover conditions.

- The redevelopment of any site not currently served by water quality best management practices shall achieve at least a 10 percent reduction of nonpoint source pollution in runoff compared to the existing runoff load from the site. Post-development runoff from any site to be redeveloped that is currently served by water quality best management practices shall not exceed the existing load of nonpoint source pollution in surface runoff.

The CBLAD Local Assistance Manual includes a Guidance Calculation Procedure that outlines the steps needed to determine if a BMP meets the criteria. The Guidance Calculation Procedure is included in this manual as Appendix B. Because nonpoint source pollution can include many different contaminants and compounds, the calculation procedure is based upon the "keystone pollutant" concept. The keystone pollutant is an indicator pollutant, the existence of which provides an estimate of the total level of pollution in the runoff. The keystone pollutant for the Tidewater Virginia area is total phosphorus.

When a stormwater management facility is proposed outside of a CBPA (RPA or RMA), it is recommended that the projected water quality enhancement be calculated. Although other methods can be used, the Guidance Calculation Procedure provides an estimation. It has been recognized that the CBLAD procedure should not be used without understanding its limitations and lack of historical data. Long-term monitoring of all types of structural and non-structural BMPs will allow more detailed calculations of removal efficiencies for a variety of pollutants. It is important to evaluate each situation and not to apply blanket requirements arbitrarily. This is especially critical if the procedures or methods are used for regulatory or enforcement purposes.

4.4.2.2 PHYSICAL FEATURES FOR WATER QUALITY ENHANCEMENT

The physical design of retention basins specifically for water quality enhancement is still a relatively new procedure. There are, however, some features and configurations that have been shown to provide successful results to date.

Permanent Pool Volume: The size of the permanent pool in a retention basin compared to the size of the contributing watershed is a primary factor in basin performance. While larger ponds are typically more successful, there seems to be a certain threshold size after which further water quality improvement by sedimentation is negligible. From the standpoint of time, most pollutants have settled out at their maximum level after 24 hours. After 48 hours, little benefit is gained by sedimentation.

The state Stormwater Management Regulations require that the permanent pool be at least 3 times the water quality volume for a development area in size. Other agencies and localities base their criteria on retention time or some other factor. It is recommended that the initial basin sizing conform to the State Regulations at a minimum, with further calculations and evaluations based on the desired pollutant removal efficiency.

The permanent pool needs to be sustained by low flows either generated by groundwater or rainfall. This is discussed in Section 2.2.3 of this document. If the permanent pool is at groundwater level, the seasonal fluctuation needs to be an item of concern especially if a wetlands fringe is established.

Pond Shape: Short circuiting can affect the water quality function of a retention basin. The basin, therefore, must be designed to prevent short circuiting. This is done by maximizing the length between the inlet and outlet. The recommended minimum length to width ratio for a retention basin is 3:1. Long, narrow, and irregular shapes for retention basins also reduce the surface area exposed to wind, which, especially for shallower basins, prevents the resuspension of previously settled material. For purely aesthetic effects, irregularly shaped basins also appear more natural, or less "engineered."

Pond Depth: Pond depth is an important design criteria since most of the pollutant removal is accomplished by settling. Since very shallow basins may be prone to resuspension of materials by wind or flow effects, and deep basins can be subject to thermal stratification and can release pollutants back into the water, an average pond depth of 3 to 6 feet is recommended. Deeper ponds are being used and with proper design to account for public safety and groundwater problems, they could be considered. Thermal stratification is not usually a problem until depths become greater than 30 feet.

A 10-foot wide shelf with a 5:1 slope is recommended around the perimeter of basin to provide a margin of safety for the public. If aquatic plants or a wetland fringe is provided on the bench, the depth should be about 1' or a 10:1 slope. The stability of soils should be considered.

In general, the outlet structure should be located in the deeper portion of the basin so that the basin naturally drains to it and the maintenance drain can completely empty the basin if necessary.

Vegetation: The growth and establishment of vegetation around the perimeter of a retention basin can:

- enhance pollutant removal
- provide a habitat for wildlife and waterfowl
- protect the shoreline from erosion
- trap incoming sediment
- provide an environment for microorganisms that can remove pollutants from the stormwater biologically

Additional details are provided in Section 5 of this manual.

Site Requirements: Retention basins are not recommended in watersheds of less than 10 acres unless a natural spring exists on site. Maintaining a permanent pool is difficult in small drainage areas. Proposed basins in drainage areas of less than 30 acres should be evaluated carefully. A general rule of thumb is that 4 acres of contributing watershed is needed for each acre-foot of storage.

If soils beneath the proposed basin are permeable, such as SCS soil groups "A" and "B", a liner may be necessary to prevent the basin from emptying through infiltration. Liners can consist of clay or other impermeable soils or geotextile materials. Dense compaction of the soils may also be successful.

A buffer area of about a 25-foot width should be included around the retention basin. This buffer area should receive vegetation including trees and be managed as a meadow, not as a lawn. No trees, however, should be planted in the embankment structures.

4.4.3 DESIGN MODIFICATIONS FOR WATER QUALITY ENHANCEMENT

This section contains descriptions of modifications to typical retention basin design that can be made to enhance the stormwater quantity and quality control functions. The modifications primarily relate to the basin storage and release facilities. An additional modification which can be considered, especially for a retention basin, is the establishment of a wetland or shallow marsh area in and/or around the basin. These aspects are described in section 5.0. The design guidelines included for these modifications are in addition to and should be used in conjunction with the other retention basin guidelines described above.

4.4.3.1 EXTENDED RELEASE

The use of extended release in a retention basin relates directly to the stormwater quantity control function of the basin, and also impacts the water quality enhancement function. In this case, the stormwater outlet structure is designed to extend the release of the runoff flow. Instead of being located at the bottom of the basin, however, the extended release orifice is the low flow stormwater quantity control orifice, located above the permanent pool elevation.

One method of providing extended release in a retention basin is by having the low flow outlet pipe at a negative slope from the riser to the outlet structure. The pipe opening is located about 1 foot below the permanent pool surface. This keeps the floating debris from blocking the inlet pipe. The orifice can be protected by wire mesh for further protection.

4.4.3.2 WATER QUALITY STORAGE

The Commonwealth of Virginia Stormwater Management Regulations require the release of the "water quality volume" of stormwater over a minimum 30-hour time from the point of peak stormwater storage in a detention basin. However, in a retention basin the water quality storage is a permanent pool with a volume equal to three times water quality volume computed the same way as for a detention facility. That is, the volume is equal to the 0.5" of runoff multiplied by the total drain area. The flood storage above the water quality storage has no specific release time and is governed by the hydrology of the event and the hydraulics of the system.

4.4.3.3 RETROFITTING EXISTING FACILITIES

For retention basins, most retrofitting tasks involve modifying the outlet structure or improving the storage capacity of the basin. These can include:

- excavating the existing basin to create additional storage capacity;
- adding to the elevation of the embankment, also creating additional storage capacity; or
- constricting or modifying the outlet orifices, thus changing the release rates and storage configurations.

The new storage capacity can be used to improve the quantity control function by releasing flow at a lower rate, extending the release time of the stormwater runoff, increasing the permanent pool volume, creating a shallow marsh or wetland area, or a combination of all of the above.

The basic design guidelines for any of these tasks will be the same as if the basin were being constructed originally. The hydrologic and hydraulic analyses must, however, be performed with exact information concerning any limits that the existing facility configuration will have on the desired functions of the basin.

4.5 CONSTRUCTION AND OPERATIONS ISSUES

The construction, operation, and maintenance of all types of retention facilities are primary factors in their success rate and longevity. A basin can be designed utilizing proven criteria and state-of-the-art techniques, but unless it is constructed according to that design and maintained so that it continues to emulate the original design, it will not be able to operate efficiently and achieve its desired water quantity and quality control functions.

The following sections contain guidelines for successful construction, operation, and maintenance of retention basin BMPs. It is incumbent on the administering locality that these guidelines, along with other proven and accepted techniques, be adhered to throughout the operating life of the facility.

4.5.1 CONSTRUCTION GUIDELINES

The construction of retention basin BMPs must be completed both according to the original design and utilizing practical knowledge about the intended function of the facility. The following guidelines include techniques, methods, and recommendations formulated from construction and operation experience.

Construction Schedule: Because it is typically a portion of the site utilities, and it is often used as a sediment basin during the construction of the upstream development, a retention basin should be one of the first facilities planned and constructed on the site. Additionally, any temporary drainage or erosion control facilities should be constructed during the initial phases. Temporary facilities can most efficiently be used to keep stormwater or erosion and sediment damage from occurring to the permanent facilities.

Site Layout and Preparation: The outside perimeter of the retention facility should be staked out before any clearing and grading begins. The embankment and any appurtenant work like stream bank stabilization should also be staked at this time. In general, at a minimum the following layout stakes, marked for grade, should be used:

- top of slope of the basin excavation
- bottom of slope of the basin excavation
- centerline of embankment
- front and back toe of slope or embankment
- several grade stakes through the basin floor

The first stakes to be set should be the centerline of the embankment and the top of the slope of the basin excavation.

The outlet control structure should be staked, constructed, and backfilled before general earthmoving is started.

The site must be dry for successful excavation to take place. If a site is wet, or if the site is expected to be wet during construction, measures should be taken to ensure proper conditions. These measures could include direct drainage trenches to points of lower elevation or the collection of runoff and surface water in sumps that require pumping. If a typically wet site has been selected for ease in maintaining the permanent pool, appropriate dewatering techniques should be used.

<u>Embankment Construction</u>: Good fill material, suitable soils, and proper compaction techniques are imperative for the construction of a stable embankment. Increasing the embankment breadth and decreasing the slope can also be important measures.

Placing embankment fill should be performed in sequential lifts of 6- to 8-inches each. An entire lift across the embankment should be completed before the next lift is begun. This allows any moist soils to dry and additional compaction to occur from the application equipment.

The proper construction of a cutoff trench is imperative to prevent any undermining of the embankment. A cutoff trench is a trench excavated along the centerline of the embankment before the fill materials are placed. It must be constructed from a relatively impermeable soil. The cutoff trench can be constructed wide enough for the bulldozer or other equipment to work within it. The impermeable soils should be placed in 6- to 8-inch lifts. The cutoff trench must extend from several feet below the existing grade up into the embankment fill.

The primary location of concern in terms of potential embankment failure is the point where the outlet pipe passes through it. The placement of antiseep collars to prevent soil piping failures is of key importance. An antiseep collar is a metal, concrete, or masonry shield which is placed around the pipe within the fill embankment. The backfill material around the outflow pipe should also be properly placed and compacted to help prevent embankment failure.

<u>Inflow and Outflow Structures</u>: The inflow and outflow control structures must be constructed correctly in order for the basin to operate as intended and designed. Both structures require accurate surveys to ensure they are placed at the correct elevations.

The inflow structure is generally less critical than the outflow structure, but it still requires accurate vertical placement. Slope protection should be used ahead of a stream inflow structure, downstream of an inflow "spillway" of any length, and at the basin discharge point. The inflow control structure must be constructed so that it directs the flow into the basin forebay, or across the basin floor as intended by the basin design.

The outflow structure may contain several key components that must work in concert with each other. These may include weirs, orifices, grates, or other flow control sections. These must all be properly constructed and placed at accurate elevations. If the structure is constructed offsite, it must be inspected carefully upon delivery to the site for any defects or misalignments of any of the components. The final placement and/or construction must be exactly as shown on the construction documents.

Construction Operations: A retention basin is most subject to externally caused damage during its construction. Since most basins will be located at the low points of a site, they must be protected from extreme rainfall events that may occur during construction. Vegetative cover and the emergency spillway must also be completed as quickly as possible during the construction phase.

The use of an inspector is one of the best method of ensuring that the detention basin is constructed as designed. This inspector may an in-house representative, someone from the designing firm, or from an outside consultant or inspection company. The inspection may be full time or part time. The primary focus of the inspections should include:

- embankment fill placement
- embankment fill material
- implementation of adequate erosion and sediment control

Additional details can be found in Section 6 of this document.

4.5.2 COST ESTIMATES

The graph in Figure 4-3 shows the average cost to construct a retention basin of a size which would be required for a given total project area. Likewise, the graph in Figure 4-4 compares the construction cost associated with the volume of stormwater which must be detained. These figures are generic and should only be used as guidelines.

The pre-development C-factors used for the associated hydrographs ranged from 0.35 for a small 1-acre site to 0.20 for a larger 50-acre site. Post-development C-factors were determined by the types of development typically associated with various sizes of land. These post-development C-factors ranged from 0.80 for small sites to 0.50 for larger sites. Retention for the water quality volume was also included in these estimates, as was the permanent pool in a retention facility with a volume equal to three times the water quality volume. The costs of constructing retention facilities were generated from recent contractor estimates of various basin sizes and were compared with Means Sitework and Landscape Cost Data, 1991.

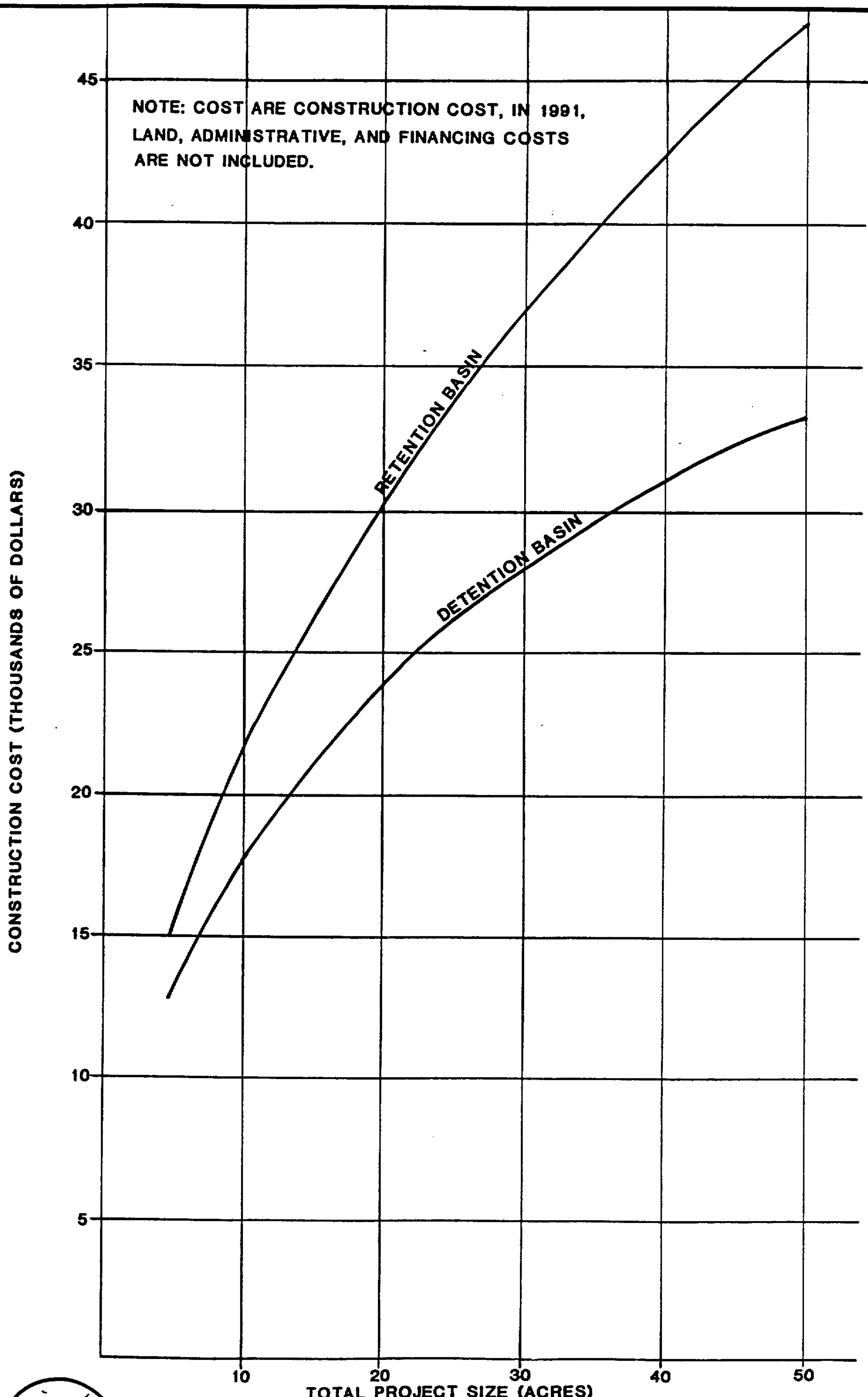

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FIGURE 4-3
TOTAL PROJECT SIZE VS. FACILITY COST

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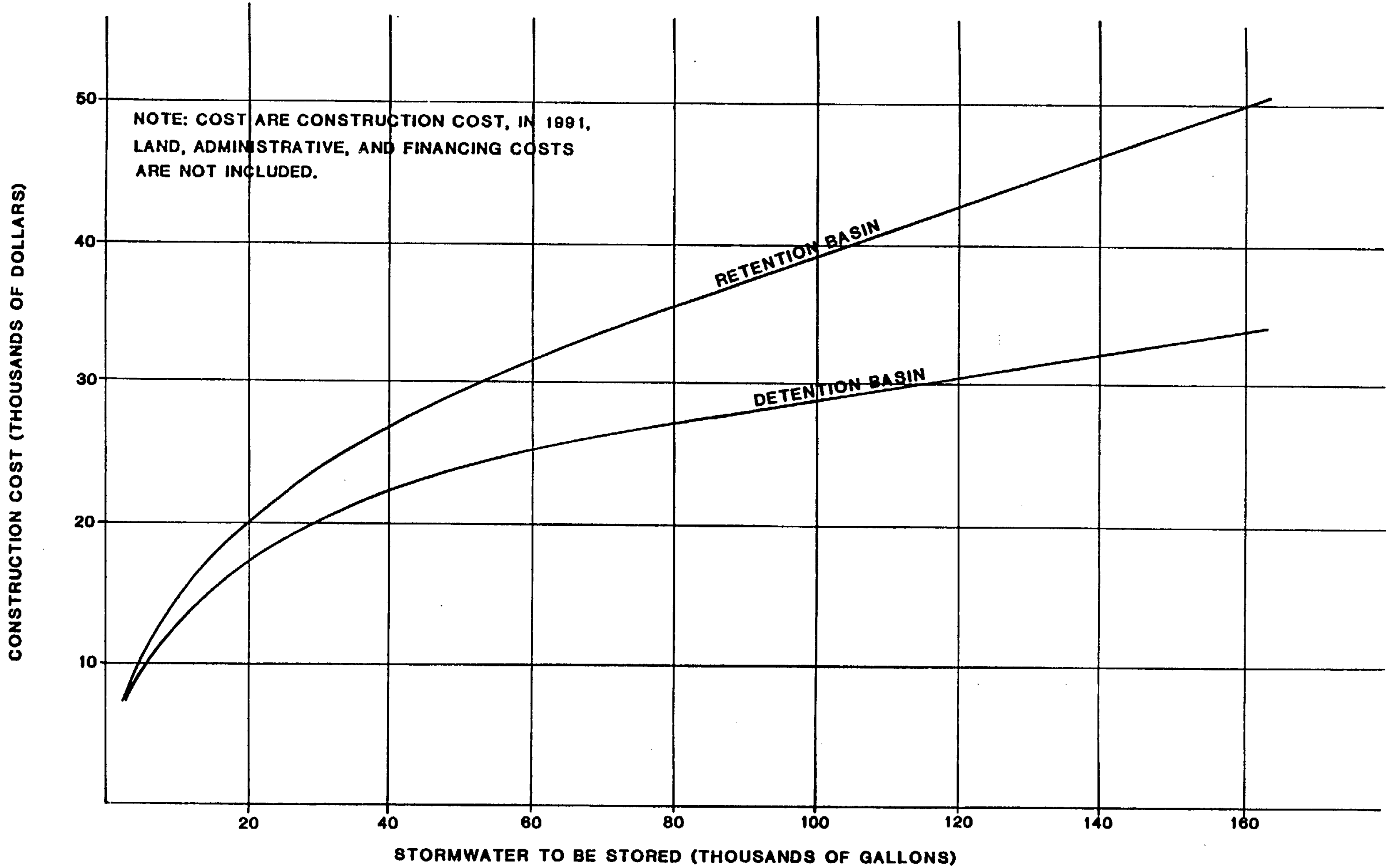

FIGURE 4-4

HAMPTON ROADS PLANNING DISTRICT COMMISSION

CONSTRUCTION COST VS. STORMWATER TO BE STORED

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4.5.3 FACILITY LIFE EXPECTANCY

The life expectancy of a retention facility is directly proportional to the quality of construction and the maintenance. If properly placed in stable soil, concrete can be expected to last up to fifty years. Metal facilities can be expected to last twenty years or more if properly maintained. Aluminum alloy products will have a longer life if properly specified. Preventative and corrective maintenance are crucial to the success of the forebay and the basin bottom.

4.5.4 MAINTENANCE REQUIREMENTS

The agency responsible for long term maintenance must be identified during the planning stages. Even though a retention facility only performs its design role on an occasional basis, it must be constantly prepared to do so. A comprehensive, regularly-scheduled maintenance program is the key to any successful stormwater management facility. Such a program is comprised of funding, maintenance, inspection, training and program reviews.

A retention basin will be useless if funding for its maintenance is not adequate. Funding considerations include: staffing, equipment, and material needs; facilities for storage of materials; storage, maintenance and replacement of equipment; training and administrative costs; seasonal effects; long-term capital improvements; and emergency appropriations for unforeseen problems.

The physical portion of the maintenance program should include aesthetic, preventive and corrective measures.

Regular, major, and informal inspections should be performed. Major inspections should be performed semi-annually and after each major storm. Regular inspections should be conducted to determine the need for and the effectiveness of maintenance work. Informal inspections should be conducted during every visit to the facility by maintenance personnel, and, if possible, prior to the occurrence of a major storm. Further details can be found in Section 6 of this manual.

A training program should include: maintenance and inspection techniques; proper record keeping,and stormwater program goals and objectives. Particular attention should be paid to the purpose and operation of stormwater management facilities, the importance of thorough maintenance, and the health, safety and other consequences of maintenance neglect.

Additional information can be found in Section 6 of this document.

4.6 PLAN SUBMITTAL REQUIREMENTS

Submittals for detention facilities need to be made to the locality in accordance with local ordinances and regulations for Stormwater Management, Erosion and

Sediment Control, and Chesapeake Bay Preservation Ordinances. Only State agencies need to submit plans to the Division of Soil and Water Conservation and the Chesapeake Bay Local Assistance Department.

4.6.1 AGENCIES

The Department of Conservation and Recreation Division of Soil and Water Conservation can provide assistance on the Erosion and Sediment Control Law, Stormwater Management Act, and Dam Safety Act and Regulations.

The State Water Control Board, Permits Section can provide information on the stormwater NPDES permit.

The Chesapeake Bay Local Assistance Department can provide information on the Chesapeake Bay Preservation Act and Regulations.

The Norfolk District Corps of Engineers Construction Operations, Regulatory Permit Section, issues permits for wetlands disturbance and navigable water crossings.

VMRC and local Wetlands Boards need to be contacted for wetlands or projects impacting the shoreline.

If a permit is needed for construction because of wetlands disturbance or interference with a navigable stream, then a permit would probably be needed for maintenance dredging. If no permit is issued or needed for construction, then no permit would probably be needed for maintenance dredging. Dredge materials under either circumstance would probably not be regulated. At this point in time, there are no requirements for disposing of materials dredged from retention or detention basins; however, it would be prudent to discuss the issue with the Corps of Engineers prior to dredging.

4.6.2 SUBMITTAL CHECKLIST

A checklist has been prepared for overall guidance and can be found in Appendix A. Localities may have their own checklist which the applicant would need to follow. In addition, the Division of Soil and Water Conservation has developed a checklist to be used by State Agencies.

5.0 ESTABLISHMENT OF WETLANDS IN STORMWATER DETENTION OR RETENTION BASINS

Well-planned stormwater management practices will reflect the natural processes of the environment. The artificially-created or planned fringe wetland, in a basin designed to also provide stormwater control, makes practical sense.

The design recommendations described within this section are geared toward a functional use of available resources. The wetland basin can be part of a broad planning approach which combines many values and uses within an area traditionally considered wasted space. Therefore, the most desirable approach is to provide a balance of functions and not strictly "wetland creation" or "mitigation effort." A clearly stated purpose and operating plan during planning and design will address the future maintenance needs and purpose of the basin, which is ultimately stormwater control.

The establishment of wetlands in stormwater detention or retention basins is another step in providing multi-objective functions. The benefits of creating wetlands in new or existing detention basins include improved water quality of stormwater discharges, increased urban wildlife habitats, and improved environmental awareness in the local communities. The success of the wetland basin depends on long-term commitment to monitoring and maintaining the facility. The most effective way for local jurisdictions to manage these facilities is through community cooperation in a joint 'project team' relationship. Local subdivision organizations, clubs, scout troops, or environmental action groups serve as project sponsors to provide routine maintenance and watch dog services, while the local jurisdiction provides periodic maintenance.

Stormwater basins can be attractive, functioning wetland habitats. Standard basin design is generally not appropriate for wetland establishment, but creative designs that consider both the criteria for stormwater management and the wetland establishment are possible. The driving force behind any wetland is the hydrology. If the appropriate conditions in the basin can be established and maintained through the growing season, then wetland creation in the basin is possible. Care must be taken to choose plant species that are compatible with stormwater basin functions. Above all, an aggressive maintenance program should be instituted and adhered to.

A wetland basin should always be considered where the proper hydrologic conditions are present, sufficient land is available for creation of the basin, and 404 Permit requirements for subsequent maintenance or expansion of the facility are not too restrictive.

TABLE 5-1
BENEFITS OF ESTABLISHING WETLAND BASINS

<u>Socioeconomic Values</u>
- Flood control
- Erosion control
- Groundwater recharge and water supply
- Fishing
- Recreation
- Aesthetics
- Education and scientific research

<u>Fish and Wildlife Values</u>
- Fish habitat
- Waterfowl and other bird habitat
- Furbearer and other wildlife habitat

<u>Environmental Quality Values</u>
- Water quality maintenance:
  - Pollution filter
  - Sediment removal
  - Oxygen production
  - Nutrient recycling
  - Chemical and nutrient absorption
- Aquatic productivity
- Microclimate regulator
- World climate (ozone layer)

5.1 GOALS AND CRITERIA OF SUCCESS

In the initial stages of planning, the decision can be made to create a new wetland or establish a wetland in an existing basin being retrofitted. This decision will be made based upon defined goals that the basin system should be expected to accomplish. These goals must be realistic, reflect the nature of the surrounding community, and achieve success measurable against set criteria of performance. The nature of the facility and the criteria of performance will be defined by five major issues.

1. <u>Stormwater Control</u>: Stormwater control will be the primary goal of the basin. The basin may be required for floodwater storage. The need for stormwater control probably will have been previously established, and will drive the nature and size of the facility.

2. <u>Water Quality</u>: The water quality of stormwater runoff is increasingly a major concern to local municipalities. A wetland basin can significantly improve the water quality of the stormwater runoff.

3. <u>Wildlife Habitats</u>: Wetland basins will naturally become wildlife sanctuaries. The type and quality of the habitat can be controlled by design considerations.

4. <u>Community Impacts</u>: The type of wetland basin implemented must reflect the neighborhood and community wherein it is to be placed.

5. Subsequent 404 Permit requirements must not prevent adequate maintenance or improvement of the facility for its primary purpose -- stormwater management.

Some of the parameters that will influence the goal setting procedure are:

- What are the drainage requirements?
- Will the facility be a retention or detention basin?
- Is the topography of available land compatible with the proposed basin?
- Will the facility serve as a wildlife area?
- Is the neighborhood compatible with the establishment of a wetland basin?
- Will runoff conditions negatively impact the wetland basin?
- Are there special maintenance concerns?
- Is community involvement expected?

These parameters can be rated to develop the basis of design for the wetland basin. Additionally, long-term monitoring needs to be addressed and included in the design methodology in order to judge the success of the basin, and provide corrections as necessary. Figure 5-1 shows the inter-relationship of the design parameters. Figure 5-2 illustrates the target functions of the basin, stormwater control, water quality, and wildlife habitats.

5.1.1 WATER QUALITY

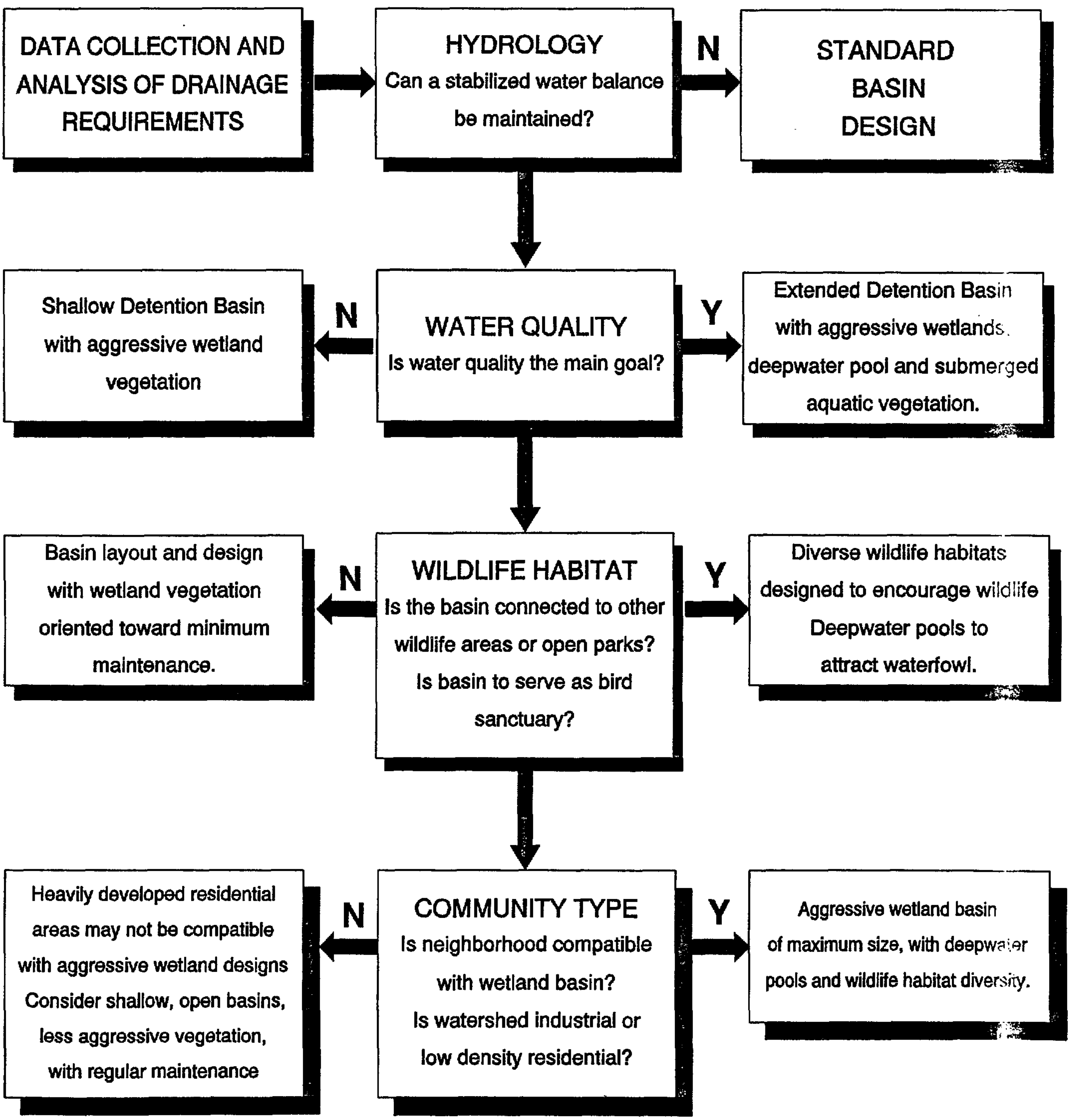
Wetland detention basins can significantly improve overall water quality of stormwater runoff. The basin will take advantage of physical, chemical, and biological processes to remove dissolved as well as suspended pollutants. Sedimentation of suspended solids, uptake by algae and rooted wetland plants, oxidation of organics, and absorption of nutrients and heavy metals will all occur within the wetland basin.

5.1.2 WILDLIFE HABITAT

The creation of the wetland basin can have a secondary benefit in creating a habitat for wildlife. A major concern of the continuing loss of wetland areas is the loss of wildlife habitat. Drainage basins and drainage channels can become part of a successful urban wildlife management plan. The incorporation of islands or pockets of wildlife habitat with corridors connecting to a 'core' wildlife area provides a vital step in restoring wildlife habitats. The corridors can be drainage channels, power lines, or utility easements. The core areas are parks or other open area such as swamps or larger wetland areas. The shallow water of the wetland basin will become the home to a variety of animals and insects, increasing the available food chain. The wetland basin can become a classroom for school children in the community, increasing their awareness of the delicate structure of the environment.

FIGURE 5-1

DESIGN PARAMETERS FOR WETLAND BASINS



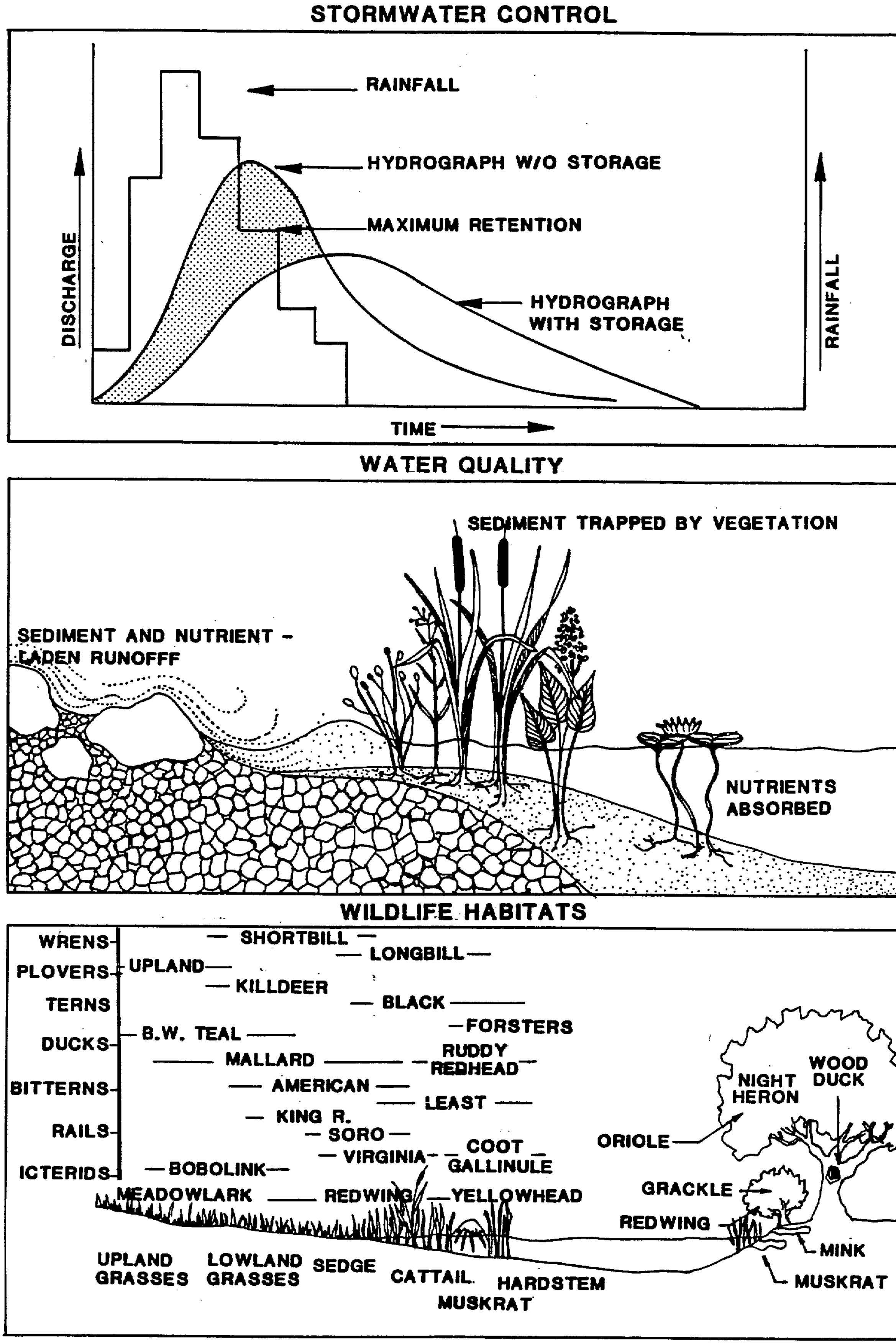


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FIGURE 5-2

TARGET FUNCTIONS

Source:
Burke, et. al.,
Protecting Nontidal Wetlands, 1988.

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5.2 IMPLEMENTATION GUIDELINES

5.2.1 REQUIRED DATA

The following will be required in the development and design of a stormwater wetland basin.

- Soil borings (classification of subgrade)
- Exfiltration (percolation) tests
- Seasonal groundwater elevations
- Drainage area (size in acres)
- Land use characteristics
- Discharge conditions
- Inlet invert
- Topographic map of proposed site

5.2.2 PHYSICAL ASPECTS OF BASIN

The physical aspects of the wetland basin are; the size of the wetland area, the size (shape and depth) of the deeper water pools in a retention basin, control of inlet velocity, control of short-circuiting, the outlet structure, and the water balance of the wetland.

5.2.2.1 WATER BALANCE

The most important aspect of establishing the wetland basin is adequate control of the water balance. Most wetland plants have narrow water depth tolerances. Properly functioning basins will often have sufficient water levels in the spring, but during drought periods, water levels drop drastically. It is crucial to verify that pool elevations can be maintained during drought periods.

Water inputs can include stormwater runoff, base flow, and groundwater. In the Tidewater region, seasonal groundwater elevations can vary widely. Groundwater infiltration can produce a basin which has an unacceptable depth variation to the narrow water depth requirements of most wetland plants. Most natural upland wetlands are standing depressions with an impervious substrata. This can be artificially replicated in a designed wetland basin by using a clay layer, or a plastic pond liner.

Recognizing that wetland vegetation has a narrow range of water depth tolerances, it is critical to calculate properly the water balance. Yearly, or average, total water budget calculations for the basin are not sufficient to determine whether water will be present in the basin during drought periods. Outlet structures which are designed to hold water longer than 24-30 hours which will flood and cover wetland vegetation can kill the wetland plant material. The important calculation is to determine the wetland area which can be maintained during low, dry summer month flows. It is also important to control how long water will be detained at elevations which will totally flood the plants.

Most evaporation calculations rely on deep water as a determining factor. Shallow water is generally warmer and evaporates faster. In addition, the wetland basin will be vegetated, so transpiration must be considered. Data on evaporation and monthly rainfall can be found in Section 2.2.3. Figure 5-3 illustrates water balance calculation.

5.2.2.2 WETLAND SIZE

It is recommended that as much of the basin bottom as possible be utilized for the wetland creation and still meet the other multi-purpose objectives. The contribution of the wetland to water quality enhancement will vary with the surface area available for the wetland because increased wetland area improves vegetative uptake and solids settling characteristics. For those basins that use large increases in depth, instead of surface area, to control peak flows, the wetland will not contribute as significantly to water quality. The greatest benefit to wildlife and pollution control is achieved when the maximum area is used for the shallow wetland basin.

Although it is strongly recommended that wetland basins be used in conjunction with extended release detention, that may not always be possible. If not, it is recommended that the surface area of the wetland account for a minimum of 3 percent of the area of the subwatershed draining into it.

5.2.2.3 WETLAND PERMANENT POND:

1. Frequently Flooded Areas: As mentioned above, the surface area of the wetland should be maximized in relation to the surface area of the entire stormwater basin. However, all wetland basin facilities should include a transitional area that is not entirely wetland and not entirely upland. This is part of the flood control and extended release volume or bordering areas that will be flooded whenever stormwater runoff enters the basin, but that will not contain standing water after the extended release period. This area will support a diverse group of plants that will thrive on the damp soil and which can tolerate the brief periods of flooding associated with extended release detention. However, most of these species would not be likely to tolerate constant inundation. The plant community in this area will provide cover and a food source (seeds) for certain nesting bird species. The transition zone should include the area within 10 to 20 feet from the edge of the permanent pond.

2. Shape and Depth of the Permanent Pond: Shallow water (i.e., < 12 inches) promotes the growth of most species of emergent wetland vegetation. Since emergent vegetation is a contributing factor in the pollution removal capability of the wetland, and provides value to wildlife, the water of the wetland should be

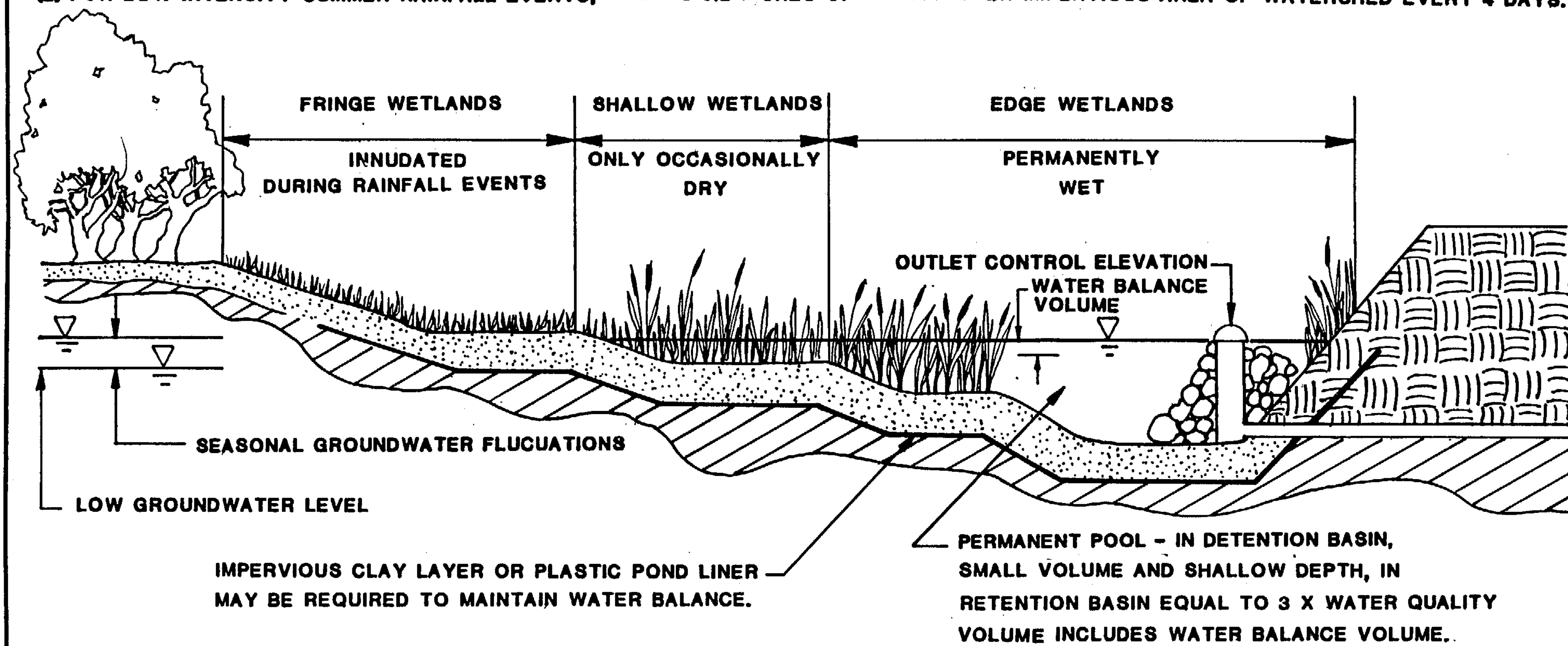
WATER BALANCE

CALCULATED DURING DROUGHT CONDITIONS[1]

BASE FLOW + GROUNDWATER GAIN + LOW INTENSITY RAINFALL[2] =
EVAPORATION + TRANSPORTATION + INFILTRATION

(1) AT OTHER TIMES, EXCESS FLOW ABOVE OUTLET CONTROL ELEVATION WILL BE DISCHARGE.

(2) FOR LOW INTENSITY SUMMER RAINFALL EVENTS, ASSUME 0.2 INCHES OF RUNOFF OVER IMPERVIOUS AREA OF WATERSHED EVERY 4 DAYS.



HAMPTON ROADS PLANNING DISTRICT COMMISSION

FIGURE 5-3
WETLAND COMMUNITIES AND WATER BALANCE

URS CONSULTANTS

limited to the depths conducive to the growth of emergent vegetation.

Approximately 75% of the wetland should have water depths less than 12 inches and 25% of the wetland should have depths ranging from 2 to 3 feet. The deeper depths will result in open water, which will make the entire wetland more attractive to waterfowl.

Waterfowl seem to prefer a habitat with both cover and open water (Weller 1978). In addition, the deeper water will favor the growth of submerged aquatic vegetation, another food source for waterfowl. It is important to note that it may not be appropriate to attract waterfowl to the wetland basin if water quality is an overriding concern. This is because waterfowl can add excessive nutrients and bacteria to the water column with their excrement. The additional nutrients promote algal growth, which in turn causes eutrification of the system. Eutrification leads to oxygen depletion and poor water quality.

The deeper area of the wetland should include an outlet structure design such that sediment does not block the outlet pipe. A basin forebay should be established at the pond inflow points to capture larger sediments.

After passing through the forebay, the incoming runoff should pass through shallow areas of emergent vegetation in order to maximize sedimentation and the mixing of runoff with the shallow pond water. As much vegetation, and as much distance, should separate the basin inlet from the outlet as possible as discussed in Sections 3 and 4 of this document. This will avoid a "short-circuiting" effect whereby stormwater runoff flows out of the wetland with only minimal treatment by the wetland.

Seventy-five percent of the wetland should be 12 inches deep or less. Of this area one third should range from 6 inches deep to 12 inches deep, and the remaining two thirds should be 6 inches or less in depth. The water depth should slope gradually but regularly from the 12 inches depth at the edge of the deep area to the 6 inches depth, and then fix the basin depth at roughly 6 inches throughout most of the remaining two thirds of the shallows. The water should gradually get shallower about 10 feet from the edge of the pond.

It is necessary to have precise depths and a uniformly graded substrate. Grading also will "soften up" the basin soil enough that no supplemental disking or plowing will be required. In retrofitting existing facilities, if the basin does not need grading or excavating the soil should be broken up with a disk or chisel.

Several wetland layouts are possible. Basins can have the inlet and the outlet at opposite ends of the basin. If this is not possible the incoming stormwater runoff should be channeled away from the outlet structure into the stand of emergent vegetation.

5.2.2.4 MODIFICATIONS TO THE INCOMING FLOW OF WATER

As discussed above, if the outlet and the inlet structures must be located close together it is recommended that the incoming stormwater runoff be channeled away from the outlet structure. If runoff enters the wetland with high velocity, bottom scour and damage to the emergent vegetation could result. If high water velocity is a potential problem, some type of energy dissipating device should be installed. For channel flow rip-rap placed along the channel should be installed as needed. For basins in which runoff enters directly through a pipe the runoff should be directed at an energy dissipating structure. This would be a particular requirement for inflow pipes which discharge from above the surface of the wetland.

5.2.2.5 OUTLET STRUCTURE AND EXTENDED DETENTION

The requirements of an outlet structure for a wetland basin include damming up the volume of water needed for wetland creation, detaining a certain volume of water for extended periods of time, and permitting water to flow from the wetland without blockage. Outlet structures include the common barrel and riser type and a simple damming device for retrofitting existing peak flow attenuation devices. The structures operate by damming up the base flow that previously left the basin at ground level. The structures have an orifice (or orifices) that permits the runoff to leave the basin. Above the orifice is a section of the outlet structure that detains a certain volume of runoff during storm events as runoff enters the basin faster than it can leave the basin through the orifice. Once the storm event has ended, the water level in the basin drops as the detained runoff flows out of the basin through the orifice.

The Washington, D.C. area portion of the U.S. Environmental Protection Agency's Nationwide Urban Runoff Program (MWCOG 1983) demonstrated the value of "extended detention for dry stormwater basins" or extended release from detention basins. The low volume of permanent water storage in shallow wetlands in comparison to deep ponds and lakes makes it important to incorporate extended detention into the wetland basin design. Based upon previous studies, it is recommended that the runoff from a one year storm be detained for at least 30 hours for wetlands design considerations.

The orifices used for extended detention will be vulnerable to blockage from plant material or other debris that will enter the basin with stormwater runoff. Some form of protection against blockage will be necessary. The device used to protect the orifice must allow water to pass freely through the orifice, therefore some type of non-corrodible wire mesh is recommended. Wire mesh laid directly across the orifice will not suffice since the mesh can become blocked almost as easily as the orifice itself. Instead, the mesh should have some depth to it, so that the area that must be blocked in order to interfere with water flow is relatively large. Given enough time, most of the floating organic materials (e.g., wood, vegetation, paper) will lose buoyancy and sink. Hemispheric, pyramid and box-like structures are possible, among others. The devices should be made of mesh and reinforced so that they do not collapse and lose their functional surface area. They also should be attached very securely to the outlet structure. Routine maintenance of the protective device will be required. The smaller the mesh on a protective cover, the more efficient it will be at protecting the outlet structure, and the more likely it will clog. Design also should take into account possible damage and ice and freezing of the strainer device.

5.2.3 BIOLOGICAL ASPECTS OF THE BASIN

5.2.3.1 WETLANDS SUBSTRATE

Wetland soils that have supported wetland vegetation will generally contain a pool of plant propagules. This type of soil can prove valuable when constructing an artificial wetland, since many of these propagules can be expected to generate vegetation in the new wetland. However, it is unlikely that a site for an artificial wetland will contain a preponderance of wetland soil, or that such soil will be valuable for application to the basin. On the other hand, it is likely that dry basins scheduled for conversion to wetland basins will contain at least isolated pockets of such soil. These basins will have base flow, and the presence of base flow generally results in wetland vegetation. The vegetation may be located along the channel that contains the base flow, or there may be pockets of wetland vegetation in low areas of the basin that do not drain completely.

Opinions differ as to whether this type of soil should be preserved when the wetland basin is created. If excavating is necessary the wetland soil should be saved and then spread over the graded basin in the areas planned for shallow water only if it can be done without stockpiling. Stockpiling and mechanical handling of the existing wetland soil will have detrimental biological effects on the soil. However, the best choice would be to keep the wetland soil in its present position with no disturbance, although this will only be effective if the elevation of the wetland soil is compatible with the final elevation of the permanent pond.

Most of the soils that will be available for constructing artificial wetlands in stormwater basins will be acceptable for the establishment of wetland vegetation. Soil depth may be more important than soil type in establishing vegetation, since the plants must be anchored securely to the substrate to avoid being uprooted. A soil depth of at least 8-18 inches is recommended for the shallow wetland basin. If there is insufficient substrate depth on the basin, the remainder can be made up using sand or other available soil material.

5.2.3.2 WETLAND VEGETATION

1. <u>Planting Wetland Vegetation</u>: The long term effects of artificially established wetland vegetation on the vegetative development of the wetland is not yet clear.

However, data show that artificial establishment does influence the short-term development of the wetland. In addition, volunteer species are not likely to occur in large numbers for at least several years after establishment of the wetland. For this reason, it is recommended that wetland vegetation be artificially introduced on newly constructed wetland basins.

2. Desirable Species Characteristics: Shallow wetlands will not be completely homogeneous even if the construction plans call for homogeneity. Differences in soil types, depth, water circulation, and other aspects will result in habitat variations on the wetland. To take advantage of this heterogeneity, more than one species of wetland vegetation should be established on the wetland. A greater number of sites on the wetland thus can be utilized, which will increase the probability that more of the wetland will be vegetated. There are other benefits also, including an increase in the diversity of food for wildlife, which will likely result in utilization by more wildlife species. In addition, the growth cycles of wetland species differ, with some species reaching peak biomass in the late spring while others will not reach peak biomass until later in the summer. A mix of such species will result in maximizing the presence of vegetation on the wetland throughout the growing season.

The most important species to be established on the wetland are the ones that spread aggressively. Aggressive species will spread to other sections of the wetland and by their increase in numbers make it more likely that the establishment will be successful. In addition, rapidly spreading species will make it unnecessary to plant vegetation in all parts of the wetland, thereby resulting in a savings of money and manpower.

It is recommended that all the aggressive wetland species (which will be termed "primary" species here) be established in quantity on the wetland. This should ensure that the artificially established vegetation will spread and influence the species composition of the wetland for several years.

In addition to the primary species, it is recommended that 3 additional species (termed "secondary" species) be planted on the wetland, although in far lesser numbers than the primary species. These species should have wildlife, aesthetic and other values but need not be as aggressive as the primary species. This small pool of secondary species may provide the wetland with larger populations of these important species in later years.

3. Selection of the Wetland Species: The primary qualities that are needed in a species for establishment on an artificial wetland are aggressiveness in spreading and value to wildlife. Although the productivity, and thus the nutrient uptake of species may be important to water quality improvements, there is not yet enough information available to select species by nutrient uptake rates.

Not all species have both aggressiveness and value to wildlife. Two species, Typha spp (cattail) and Phragmites australus (common reed) are aggressive wetland species but do not appear to have good wildlife value. This is particularly true of Phragmites. Other species such as Peltandra virginica (arrow arum), have good wildlife value but are reported to spread slowly on the wetland. In Peltandra this is probably a result of very little vegetative propagation with the preponderance of spreading the result of seed germination. This does not mean that the species of wetland plants that are not rapid spreaders are unsuitable for artificial wetlands. Such species may be quite valuable to the long-term vegetation of the wetland and may even come to dominate it. However, they may not be suitable for establishing vegetation on the wetland quickly.

Two species, Typha spp (cattail) and Phragmites australus (common reed) are special cases. These species may be the best choice for water quality concerns, but, they are very aggressive and may completely dominate a wetland. Their low wildlife food value and the density of the plant communities they form can result in a wetland with a low value to wildlife in terms of both food and habitat. The high biomass production of these plants will quickly 'fill-in' a wetland, thereby reducing storage volume and increasing maintenance. It is recommended that cattail and common reed not be planted on artificial wetlands.

Non-persistent, perennial, herbaceous vegetation is probably the best plant material for basin use. The reasons for this are as follows:

1. Many of these plants are good colonizers, and are efficient at removing nutrients from the water.

2. The above ground parts of the plants decompose rapidly in the Fall, and are exported from the basin. The decomposed material has less of a chance of clogging the outlet structure. The leafy, aboveground material which is exported is rapidly decomposed and may provide food sources for downstream aquatics. Harvesting of aboveground plant materials may be feasible in some cases, but extreme disturbance of the root mass should be avoided.

3. Since the above ground material is exported from the basin, less material accumulates, and problems associated with plant material changing the volume capacity of the basin is reduced.

A recommended plant list is as follows:

Primary Species	Depth (in feet)	Available Commercially
Sagittaria latifolia (duck potato)	0 - 2.0	yes
Peltandra virginica (arrow-arum)	0 - 1.0	yes
Pontederia cordata (pickerel weed)	0 - 1.0	yes

<u>Saururus cernus</u> (lizards tail)	0 - 0.5	yes
<u>Scirpus americanus</u> (S. pungens, common three-square)	0 - 0.5	yes

<u>Secondary Species</u>	<u>Depth (in feet)</u>	<u>Available Commercially</u>
<u>Acorus calamus</u> (sweet flag)	0 - 0.25	yes
<u>Cephalanthus occidentalis</u> (button bush)	0 - 2.0	yes
<u>Hibiscus Moscheutos</u> (rose mallow)	0 - 0.25	yes
<u>Leersia oryzoides</u> (rice cutgrass)	0 - 0.25	yes
<u>Scirpus validus</u> (softstem bulrush)	0 - 1.0	yes

Newly established plants generally will not have good survival rates at the lowest depth ranges and generally survive better in the upper two thirds of the range. Established colonies will generally expand into the deeper ranges. Plants should be spaced 1.5' - 2.0' on center.

In the areas above pool elevation that flood during storm events, several other species should be considered for their wildlife values. The plants listed below are plants that can tolerate dry conditions with periodic episodes of inundation. All are available commercially. This list contains suggested species but is not exhaustive. It has to be kept in mind that these plants will contribute to the organic load of the basin, their fallen leaves can clog outlet structures, and they may increase the maintenance time needed for mowing the basin sides. All woody plants to be used as a buffer or for landscaping should be purchased in containers. Bare root plant material has a much lower survival rate than containerized stock.

<u>Fringe Species</u>	<u>Common Name</u>
<u>Panicum virgatum</u>	Switch Grass
<u>Andropogon virginicus</u>	Broomsedge
<u>Cornus</u> sp.	Dogwoods
<u>Lonicera tatarica</u>	Tartarian Honeysuckle
<u>Rosa rugosa</u>	Rugose Rose
<u>Juniperus virginiana</u>	Red Cedar
<u>Euonymous americanus</u>	Strawberry Bush
<u>Rubus</u> sp.	Blackberries

Maintenance mowing of the basin sides can be reduced by planting wildflowers or other meadows instead of lawn grasses. Generally meadows only need to be mowed once a year, but the mowed material should be raked from the site so that it does not enter the basin and clog outlet structures.

4. Number of Individuals of Each Species to Plant: There is limited information on which to base recommendations for the number of individual plants to establish on a wetland. This includes both influencing the long-term species composition and the immediate success of the artificial establishment. However, it probably can be safely assumed that if the habitat conditions on the wetland are not conducive to the growth of a certain species, that species will not do well even if planted at high population densities. Therefore, the first consideration is to have the proper water depths for the species that are going to be planted.

If habitat conditions are suitable, the entire area planned for vegetation need not be planted. Aggressive species will colonize the areas not planted. It is recommended that 30 percent of the shallow (12 inches or less) area of the basin be planted with wetland vegetation. This vegetated area should be divided into sites whose surface areas are roughly equal. Mixing species should be avoided in order to reduce competition within the planted areas. Each area should be located in that part of the wetland conducive to the growth of the species it will contain, but in addition, the areas should be placed as far apart as possible while still fulfilling the habitat requirements of the individual species. Within each area the individual plants should be spaced 1.5 - 2.0 feet on center.

In addition to these sites, small clumps of the primary species should be planted throughout the rest of the wetland to increase the probability and rate of colonization of unplanted areas. It is recommended that 40 clumps per acre per species be set out in this fashion. Each clump should contain one or more individuals of a single species. The clumps should be equally divided among the primary species. Of course, species should not be planted where water depths are not conducive to the growth of that particular species. Place the clumps throughout suitable areas of the wetland.

Based on the above discussion, the number of individuals of each species to be planted is a function of the total area and shape of the site to be planted. The planted areas should be made as square as possible within the design of the wetland, rather than long and narrow. The greater area to perimeter ratio of the square design may help to preserve the homogeneous populations of the planted species by reducing colonization.

Besides the primary species to be planted in abundance, secondary species also should be planted on the wetland. It is recommended that 50 individuals of each of these additional species be planted per acre on the new wetland. These plants should be set out in 10 clumps of 5 individuals each. The clumps should be planted within 6 feet of the edge of the pond in the shallow area leading up to the edge of the pond. In addition, the clumps should be spaced as far apart as possible, but there is no need to segregate species to different areas of the wetland.

5. <u>Depth Requirements of the Selected Species</u>: Emergent vegetation appears to grow best in water less than 12 inches deep, with depths of roughly 6 inches or less showing high growth rates. The three species mentioned above as primary species will all do well in water 6 inches or less deep. This is not to say that these species will have low survival rates in somewhat deeper water, however. They are often found in deeper water in natural wetlands, although the requirements for successful growth in deeper water may be more stringent than in shallower water. Thus, the probability of successful establishment is greater in shallower water. The plants recommended for wetland establishment are suited for the recommended wetland depths.

6. <u>Plant Propagule and Dates for Planting</u>: Wetland plant establishment using seeds has been shown to have a low success rate because of the exacting germination requirements of seeds. By contrast, growing plants and dormant underground plant parts are much more amenable to transplanting. The latter two categories of plant propagules are the primary means of plant establishment recommended here.

The growth cycle of perennial plants determines what form of propagule will be suitable for planting at particular periods during the year. At the end of the growing season, generally sometime in October or November, the above ground portion of the plant dies, while the below ground portion of the plant becomes dormant. The following spring the underground portion of the plant breaks dormancy and, using food reserves built up during the previous growing season, sends up new shoots. This growth generally begins in April.

From the above brief discussion it can be seen that the natural period of dormancy of wetland plants is also the correct time for planting dormant underground parts. Likewise, growing plants should be set out on the wetland during the growing season. In general, plants can be established on the wetland at any time of the year except late summer and fall. There are two reasons for this. First, plants must have sufficient growing time to store up food reserves in the below ground parts. Plants grown in bulk in the nursery have a difficult time doing this; thus they must be set out in the wetland sufficiently early to store up below ground reserves. It is recommended that July be the latest month for setting out actively growing nursery plants.

Dormant, below-ground plant parts are usually available in late November or December, although some dormant material may be available even earlier in the year. The primary time of the year for planting dormant material is from November through April, although actively growing material may be available in April.

Not all species are desirable to plant in the dormant state. This may be attributable to the high probability of wintering waterfowl to detect and eat the planted material, or the low percent germination of dormant material. Since these factors may vary it is recommended that the probability of successful establishment be discussed with the seller of the plant material. It is recommended that plant material be set out in the spring and summer, using actively growing material.

5.2.3.3 PLANTING PROCEDURES

1. Preparing the Site for Planting: The only site preparation that is necessary for the actual planting (besides flooding the basin) is to ensure that the substrate is soft enough to permit easy insertion of the plants. If the basin has been graded or excavated there will be no problem. However, there could be difficulty if the wetland is to be created by simply flooding a previously dry basin. Such basins may have a compacted substrate or the overlying vegetation may have formed a dense root mat or a sod that could make planting difficult. It is recommended that this type of basin be "softened up" by disking or some other type of physical disturbance before the basin is flooded. Disturbance to the upper 6 inches of substrate should be sufficient.

2. Planting Procedures: The planting procedure begins when the final site preparations have been completed. These site preparations will include the flooding of the wetland to the proper depths, or if planting is to be done on the unflooded wetland, final plans to flood the wetland as soon as planting is complete.

Dormant below-ground plant parts are easier to handle. They are stored dry, usually in mulch, at temperatures slightly above freezing to maintain the dormant state. These propagules should be kept dry until the time of planting, and freezing should be avoided. Planting consists of burying the dormant material to the proper depth in the substrate.

When growing plants are to be established on the wetland one of the primary concerns is to keep the roots moist. If the roots dry out they will be damaged and decrease the probability of a successful planting. Plants will be received from the nursery either growing in small peat pots or bare rooted. Bare rooted plants will very likely have some form of root protection when received, including moisture retaining plastic bags or water filled tubs. These plants can be kept for many days in this condition, although they should be kept out of direct sunlight. However, this type of environment is not a good one for maintaining healthy plants. They should be planted as soon as possible.

The potted plants will be received in very small peat pots that can be planted to facilitate root spreading. When the peat pot is in the hole the surrounding substrate in pressed down firmly around it.

Bare rooted plants are treated similarly to peat potted plants, although the holes should be made large enough to accommodate the roots. The roots should be spread out as much as possible when the plants are set into the hole, as opposed to being bunched together. The plants should be placed deep enough that the wetland substrate is level with the point on the plant where stem becomes root. This point is usually easily identified. Thus the final arrangement of the roots in these plants will be shallow but spread out, with an overall depth of approximately 4 to 6 inches.

If the planting is to be done before the wetland is flooded no more than 24 hours should elapse before flooding occurs if bare root plants are involved. If flooding cannot occur within that time other arrangements, such as diverting the base flow through the planted area or using water from a fire hydrant, should be employed. If only peat potted plants are involved not more than 72 hours should elapse between planting and flooding without using other means of wetting the plants.

5.2.3.4 SUBMERGED AQUATIC VEGETATION (SAV)

Submerged aquatic vegetation is an important food for waterfowl and may aid in improving the quality of stormwater runoff passing through the wetland. However, there appear to be few if any commercial SAV sources for fresh water sites at the present. Therefore, the artificial introduction of submerged aquatic vegetation is not being recommended now. However, the recommendation for a deep pond in the artificial wetland (see above) was made, in part, to provide suitable habitat for SAV. It is quite likely that SAV will be brought to the wetland by migrating waterfowl or other means.

Although the introduction of SAV is not being recommended at this it is also not being discouraged. If a source of SAV is known a small amount can be introduced into the deeper part of the wetland. Although these and most other SAV species can tolerate water deeper than the 2 to 3 feet discussed earlier, that depth range should be maintained because of the turbidity that seems to occur in stormwater basins. Deeper, turbid water might eliminate SAV habitat in stormwater basins.

5.3 MAINTENANCE AND MONITORING OF WETLAND BASINS

The following is a list of maintenance and monitoring issues which should be addressed at the design stage of implementation.

- Is there a project sponsor?
- Who will provide monitoring? How often?
- Who will provide mid-course corrections?
- How will biomass clogging and siltation be removed?
- How will nuisance plant species be controlled?
- Who will provide periodic replanting?
- Who will provide regular outlet maintenance?

There are two types of organisms that could affect the acceptance of the wetland basin by the people in the surrounding community. These include thick surface algal growths and mosquitoes. Neither of these factors are likely to be a problem in the first years of the wetland's existence. However, they could arise if sedimentation changes the flow characteristics in the basin. The primary cause of either algal mats or mosquitoes probably would be standing water in portions of the wetland that were prevented from draining properly. These areas could accumulate nutrients which would favor algal growth, while the fluctuating water level or other extreme habitat conditions could decrease and mosquitoes probably will not be a problem around artificial wetlands. The correct maintenance procedures will restore the planned drainage characteristics and very likely eliminate any algal or mosquito problems.

Solids will accumulate on the wetland by two means. These include sedimentation of suspended solids carried to the basin in stormwater runoff and base flow, and the accumulation of plant material. Accumulation of this material could result in a loss of the area of ponded water available for emergent vegetation. Two remedies are available. The first is to raise the elevation of the water level in the permanent pond by raising the height of the orifice in the outlet structure. The loss of peak storage volume by increasing the water level will be minimal considering the shallowness of the wetland. If the original design accounted for additional volume for sedimentation, then the loss is not a problem for several years.

The second remedy is to remove the accumulated solids by excavation. This will require draining some of the water from the wetland and could involve considerable disruption to the vegetative community, which could require replanting. However, much of the accumulated solids probably will be in the deeper forebay of the basin inlet where the runoff first loses velocity upon entering the wetland. This area should be easy to excavate. In addition, not all the water will have to be removed from the wetland since the overall ponded area (approximately 25% of the total wetland area) should not accumulate solids as rapidly as the forebay. Thus the deeper pond may not require excavating and can remain flooded when the shallower areas are drained.

At this time it is not clear how long shallow wetland basins will function without maintenance. To avoid maintenance as much as possible it is recommended that wetland basins be installed on stabilized watersheds and not be used for sediment control. In addition, the maintenance procedure recommended here is that the outlet structure be modified to raise the elevation of the permanent pond when solids accumulate. This procedure can be repeated until the peak storage volume requirements of the basin are in danger of being compromised, at which time excavation will be required.

MONITORING REPORT

Project:______________________ Stormwater Wetland Basin

City/County:______________________ Reviewer:______________________

Date of Report:______________________

Description of Wetland Basin:______________________

____

Stormwater Control Type:______________________

Acres Created:______________________

Monitoring Agency:______________________

Upland Type:______________________

Plant Species Used:______________________

Outflow Control Depth:______________________

Date Wetland Created:______________________

Comments:______________________

____

Monitoring and Management

Duration:______________________ Inspector:______________________

Phone:______________________ Sampling Methods:______________________

____

Stormwater Control Functions:

Sediment:______________________

Outflow Structure:______________________

Inlet Control:______________________

Wetland Functions:

Plant Communities:______________________

Nuisance Species:______________________

Fill-in:______________________

Maintenance:______________________

Wildlife Observed:______________________

Mid-course Corrections Required:

General Cleaning:______________________

Re-planting:______________________

Water Balance Control:______________________

Sedimentation Removal:______________________

Action Required:______________________

Comments:______________________

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6.0 CONSTRUCTION, OPERATIONS AND MAINTENANCE

This section has been taken substantially from the Ocean County Demonstration Study, Stormwater Management Facilities Maintenance Manual, State of New Jersey Department of Environmental Protection.

6.1 PURPOSE

This section addresses construction and maintenance procedures for stormwater management facilities (SWMF). Although the focus of this document is on detention and retention basins, the principles apply to all types of stormwater management facilities; consequently, the discussions will address these facilities (SWMF) in general. When insufficient attention is paid to these elements, retention - detention basins have resulted in poor performance as measured by water quality and flood control, as well as an increased threat to public health and safety. The guidelines are intended to be used to define minimum requirements and to assist in implementing effective and comprehensive maintenance programs. The goal of SWMFs is to mitigate the adverse hydrologic impacts of land development, protect downstream areas from flooding and erosion, and prevent stormwater-caused water quality degradation. In order to achieve the design goals, retention and detention basins require thorough maintenance performed on a regular basis.

6.2 THE RESPONSIBILITIES OF OWNERSHIP AND MAINTENANCE

The owner of a SWMF usually comes by that ownership as the result of some course of action other than a direct desire for ownership of the SWMF itself. A public agency may acquire or construct a SWMF in order to alleviate a downstream flooding condition. A private individual or corporation may construct a SWMF as a matter of necessity in order to obtain municipal and/or county approval for a development and to mitigate the project's downstream runoff impacts. In some cases, the SWMF is worked into the landscaping package, while in other cases the facility is constructed in a portion of the site with low visibility to the owner and the general public. The adage "out of sight, out of mind" often applies. The success of SWMF cannot be fully achieved without proper facility maintenance. Therefore, the owner must be aware of the facility's purpose and needs, as well as the absolute importance of proper maintenance. Failing that, the owner must be closely regulated by an agency which does.

To insure proper maintenance of the facility, the owner must have the necessary institutional, managerial, and financial resources. Even where maintenance of a private facility is enforceable by a governmental entity by means of regulations and ordinances, consistent performance of facility maintenance will not be accomplished unless the owner has adequate resources for this task.

Actual ownership of a SWMF may change throughout the life of the facility. The private individual or corporation which ultimately becomes responsible for maintenance of the facility may not be the one which originally planned, designed and constructed the SWMF. This is particularly true in subdivisions where the people responsible for maintenance were never involved in the design or construction and may have little appreciation for its purpose, function, or maintenance. When ownership of a SWMF changes over the facility's life, the success or failure of that facility is often determined before it even exists.

Therefore, basic arrangements for SWMF ownership and maintenance need to be evaluated during the planning, design, and review phases of the project. Ownership and maintenance responsibility fall into three categories:

a) public ownership with public maintenance;
b) private ownership with public maintenance;
c) private ownership with private maintenance.

Public ownership with public maintenance is the most desirable solution. This should be the goal of all localities. Charges or user fees should be established for maintaining the facilities. The stormwater utility concept is very viable and can be used in Virginia. It has been successfully used in many localities and the Hampton Roads Planning District Commission has published a report discussing the subject.

Private ownership with public maintenance should be considered in situations where the owner is unlikely to have the institutional, managerial, or financial resources to properly maintain the facility. Individual homeowners and single-family homeowners associations generally fit into this category. These groups tend to lack the incentive, knowledge, equipment and resources necessary to adequately maintain the facility. In such situations, public ownership and/or maintenance of the SWMF, as well as access to the facility and financial liability, should be considered as part of the criteria for the design of the facility.

Private ownership with private maintenance is the least desirable situation. If the private owner is concerned with the proper functioning and appearance of the SWMF and must maintain other facilities, then he is more likely to properly maintain the SWMF. Private corporations are generally capable of and willing to maintain SWMFs. Corporations are usually conscious of their public image and community status. They can be expected to have the manpower, financial resources and equipment required for proper maintenance. Likewise, condominiums and co-op apartments can also generally be considered self-sufficient, since they collect funds and maintain grounds, roads and other facilities.

Within the organization, associate, or corporation, an individual should be held responsible for the performance of SWMF maintenance. This person should be vested with sufficient authority to establish procedures and priorities for the maintenance personnel. This person should also have a thorough understanding of the purpose and function of the SWMFs along with an appreciation for the consequences of facility failure and the role maintenance plays in preventing such occurrences.

6.3 CONSTRUCTION INSPECTION

A well-designed and well-built retention - detention facility should require the least amount of maintenance effort and expense. Proper inspection is crucial to achieving that goal. Poor construction can lead to many serious maintenance and safety problems, including isolated pockets of water, slope erosion, channel scour, mosquito breeding and structural failure of dams, embankments, slopes and outlet structures.

Other SWMF maintenance problems that arise due to poor construction include:

- ground settlement
- cracked, spalled or deteriorated concrete
- incorrectly installed fittings and appurtenances
- incorrect elevations, grades and dimensions
- missing, damaged, or hidden components

6.3.1 PRECONSTRUCTION PHASE

Effective facility construction inspection begins in the preconstruction phase. The inspector should become familiar with the facility plans, specifications and other related documents. Special attention should be paid to complex components, difficult site conditions and other potential problem areas. In addition, the inspector should attend all preconstruction meetings. It is at these meetings that the inspector has the opportunity to become more familiar with the nature of the project and the key personnel involved. The preconstruction meeting should address:

1) The project's overall purpose and objective;

2) Specific areas or details of the project that are particularly complex or that otherwise require special attention;

3) Construction schedules and deadlines;

4) The establishment of a chain of command for problem solving.

A detailed list of recommended preconstruction meeting topics is summarized in Table 6-1.

6.3.2 CONSTRUCTION PHASE

During this phase, it is necessary for the inspector to understand and inspect the current construction activity . During this phase, some of the inspector's key responsibilities should include:

1) Daily Reports: A brief summary which includes all construction activity, the weather and working conditions, vehicle arrival and departure times, equipment, materials and key project personnel.

2) Shop Drawings: Shop drawings should be required for all facility components. Experience has shown that problems solved on paper prior to construction can prevent major problems later in the field.

3) Progress Meetings: These meetings afford all parties the opportunity to discuss current or anticipated problems.

4) Extra Work and Change Orders: A design which may work on paper may not always be successful in the field. The contractor may have suggestions which may aid in the progress of the work or enhance the quality of the design. The inspector should review change orders and extra work orders to determine their legitimacy. Quick resolution of these requests can improve communications, relations, and workmanship.

5) Final Inspection and Punch List: The Punch List is an effective tool that the inspector can use to help insure that all facility construction is complete and correct before final payment is made to the contractor.

6.3.3 POST-CONSTRUCTION PHASE

The new facility should be warranted by the contractor. During this time, the inspector should perform periodic inspections of the facility and immediately bring any problems to the contractor's attention. This is the last opportunity to correct construction flaws before they become the owner's maintenance problems.

Table 6-2 summarizes recommended construction inspection practices.

TABLE 6-1

TYPICAL PRECONSTRUCTION MEETING TOPICS

A. GENERAL INFORMATION

1. Attendance
2. Purpose of Project and Background Information
3. Emergency Phone Numbers
4. Construction Photograph Requirements
5. Project Sign Requirements
6. Starting Date
7. Review of Contract Documents, including Insurance Certificates, Bonds and Subcontractors Documents
8. Field Office Requirements
9. Responsibility for Notifications of Affected Property Owners and Residents
10. Chain of Command for Communications and Correspondence
11. Construction Schedules
12. Key Personnel and their degree of involvement in the Project (Inspector, Owner, Engineer, Agencies, etc.)

B. POLICE AND FIRE DEPARTMENT CONCERNS

1. Traffic Control
2. Barricades and Signs Conforming to the Uniform Manual
3. Noise Ordinance Considerations
4. Working Hours, including Weekend and Holidays
5. Vandalism and Preventative Measures
6. Flagmen and Traffic Control Officers
7. Equipment Storage and Vehicle Parking
8. Emergency Vehicle Access
9. Underground Tank Locations and Precautionary Construction Procedures
10. Storage and Use of Hazardous Materials

C. UTILITIES

1. Utility Locations and Mark-Outs
2. Coordination of Utility Relocations
3. Emergency Phone Numbers of Utility Companies

TABLE 6-1 (CONTINUED)

D. FUNDING AND PAYMENTS

1. Funding Sources and Availability
2. Procedures and Dates for Payment Estimates
3. Dates for Payments to Contractor
4. Breakdown of Lump Sum Items for Partial Payment
5. Policy for Payment for Materials on Site at the Close of a Payment Period
6. Retained Monies during and after Construction
7. Requirements of Funding Agencies

E. CHANGE ORDERS AND EXTRA CLAIMS

1. Requirements for Additional Work and Submittal of Change Orders
2. Procedures and Schedule for Review and Recommendations of Change Orders
3. Procedures for Negotiating Extra Claims and Change Orders

F. CONSTRUCTION ACCESS AND EASEMENTS

1. Easement Locations and Maps
2. Responsibility for Locating and Staking Easements
3. Available Survey Data for the Site
4. Access Requirements and Staging Areas
5. Easement Restrictions and Restoration Requirements

G. CONSTRUCTION DETAILS

1. Unique or complex Aspects of the Project
2. Testing Laboratories and Sampling Procedures
3. Cold and Hot Weather Protection Measures
4. Blasting Requirements
5. Dump Site Location for Construction Related Materials
6. Shop Drawing Requirements and Review Procedures
7. Specific Construction Techniques and Procedures
8. Review of Technical Section of the Specifications

TABLE 6-2

SUMMARY OF CONSTRUCTION INSPECTION PRACTICES

A. PRECONSTRUCTION

1. Review Purpose of the Project
2. Review Plans and Specifications
3. Obtain Pertinent Permit Documents
4. Review Permit Conditions
5. Obtain Pertinent Easement Documents
6. Review Easement Conditions and Restrictions
7. Schedule and Conduct Preconstruction Meeting
8. Obtain List of Emergency Phone Numbers
9. Obtain List of Key Personnel

B. CONSTRUCTION

1. Observe All Pertinent Construction Activity
2. Be Familiar with Construction Procedures
3. Anticipate Problems
4. Keep a Diary of all Pertinent Activities
5. Write Daily Construction Reports
6. Review Shop Drawings
7. Consult with the Contractor Frequently
8. Conduct Progress Meetings
9. Review Change Orders and Extra Claims
10. Prepare Punch List
11. Conduct Final Inspection

C. POST CONSTRUCTION

1. Perform Periodic Inspections
2. Notify Contractor of Necessary Work
3. Inspect Corrected Work
4. Prepare Record Plans
5. File all Pertinent Contract and Inspection Records

6.4 MAINTENANCE

The majority of the maintenance tasks at a well-designed and constructed retention-detention facility should be simple and routine. The physical aspect of maintenance can be broken down into three areas -- preventative, corrective and aesthetic. Preventative and aesthetic measures minimize the need for costly corrective measures. Routine tasks such as lawn mowing, maintenance and trimming keep multi-purpose facilities from becoming eye-sores and enhance their attractiveness to the general public while ensuring that they still serve their stormwater control goals. Single purpose facilities, particularly those in areas of low visibility, may not require the level of service of these preventative or aesthetic measures. The frequency of preventative and aesthetic maintenance is governed by the multiple objective of the facility and its use by the public. Corrective measures are used to rehabilitate portions of a retention-detention site which will degrade in spite of preventative measures. These three categories are described in further detail below and summarized in Table 6-3. Equipment which can improve the quality of maintenance is listed in Table 6-4. Training should be given to maintenance personnel. It increases manpower productivity and gives employees a sense of purpose in purpose in performing their tasks. This can result in more thorough, less expensive maintenance.

6.4.1 PREVENTIVE MAINTENANCE PROCEDURES

Preventive maintenance, like aesthetic maintenance, is proactive. Its purpose is to ensure that the facility remains operational and safe at all times. When conducted properly, preventive maintenance minimizes the need for costly emergency and other corrective maintenance. The following items, which are summarized in Table 6-4, should be incorporated into preventive maintenance procedures:

<u>Grass Cutting</u>: This activity should be minimized by planning to limit areas where lawn type areas are needed. Generally these are contained to areas of recreational activity. Lawn mowing and trimming can amount to 15 to 25% of maintenance costs. A regularly scheduled program of mowing and trimming of grass at SWMFs during the growing season will help to maintain a tightly knit turf, and will also help to prevent diseases, pests and the intrusion of weeds. The actual mowing requirements of an area should be tailored to the specific site conditions, grass type, and seasonal variations in the climate. In general, grass should not be allowed to grow more than 1 to 2 inches between cuttings (probably once a week during the growing season in this area). Allowing the grass to grow more than this amount prior to cutting it may result in damage to the grass' growing points and limit its continued healthy growth. Agencies such as the local Soil and Water Conservation District, Local Extension Service office and the Chesapeake Bay Local Assistance Department (CBLAD) can provide valuable assistance in determining optimum grass selections and mowing frequencies.

Grass Maintenance: Grassed areas require periodic fertilizing and soil conditioning in order to maintain healthy growth. Additionally, provisions should be made to re-seed and re-establish grass cover in areas damaged by sediment accumulation, stormwater flow, or other causes. This maintenance should be incorporated into the schedule as a spring and fall procedure.

Vegetative Cover: Trees, shrubs, and ground cover require maintenance, including fertilizing, pruning, and pest control in order to maintain healthy growth. This should be done in the spring and fall.

Removal and Disposal of Trash and Debris: A regularly scheduled program of debris and trash removal from retention and detention basins will reduce the chance of outlet structures, trash racks and other components becoming clogged and inoperable during storm events. Additionally, removal of trash and debris will prevent possible damage to vegetated areas and eliminate potential mosquito breeding habitats. Disposal of debris and trash must comply with all local waste flow control regulations. For simplicity and effectiveness, trash collection should occur on at least a weekly basis. In high visibility/usage areas, trash pick-up may need to be conducted on a daily basis. In areas of low visibility or limited access, this activity may not be required on a frequent basis. In such cases, periodic inspections should be performed to ensure that the collection is frequent enough to be effective.

Sediment Removal and Disposal: Accumulated sediment should be removed before it threatens the operation or storage volume of a SWMF. Disposal of sediment must comply with all local, county, state, and federal regulations. Only suitable disposal sites should be utilized. The sediment removal program in infiltration facilities must also include provisions for monitoring the porosity of the sub-base, and replacement or cleansing of the pervious materials as necessary. Sediment should be disposed of in accordance with State Water Control Board and Army Corps of Engineers (COE) guidelines. A dredging permit will probably be required from the COE, The Virginia Marine Resources Commission, and local Wetlands Boards if this facility lies in tidal wetlands or other Resource Protection Areas. Basins should be checked for accumulation on a semi-annual basis. Sediment removal will probably need to be performed on a bi-annual (2-year) basis.

Mechanical Components: Valves, sluice gates, pumps, fence gates, locks, and access hatches, should remain functional at all times. Regularly scheduled maintenance should be performed in accordance with the manufacturers' recommendations. Additionally, all mechanical components should be operated or exercised at least once every month to assure their continued performance.

Elimination of Potential Mosquito Breeding Habitats: The most effective mosquito control program is one that eliminates potential breeding habitats. Almost any stagnant pool of water can be attractive to mosquitoes and the source of a large mosquito population. Ponded water in areas such as open cans and bottles, debris and sediment accumulations, and areas of ground settlement provide ideal locations for mosquito breeding.

Pond Maintenance: A program of monitoring the aquatic environment of a permanent pond should be established. Although the complex environment of a healthy aquatic ecosystem will require little maintenance, water quality, aeration, vegetative growth, and animal populations should be monitored on a regular basis. The timely correction of an imbalance in the ecosystem can prevent more serious problems from occurring. Additional information on pond maintenance can be obtained through agencies such as the U.S. Fish and Wildlife Services, State Water Control Board, VIMS, and others.

Inspection: Regularly scheduled inspections of the facility should be performed by qualified inspectors. For multi-objective facilities, this should be done on a weekly basis. Single-purpose facilities should be inspected quarterly and after each major storm. The primary purpose of the inspections is to ascertain the operational condition and safety of the facility, particularly the condition of embankments, outlet structures, and other safety-related aspects. Inspections will also provide information on the effectiveness of regularly scheduled Preventative and Aesthetic Maintenance procedures, and they will help to identify where changes in the extent and scheduling of the procedures are warranted. Finally, the facility inspections should also be used to determine the need for and timing of Corrective Maintenance procedures. It should be noted that, in addition to regularly scheduled inspections, an informal inspection should be performed during every visit to a SWMF by maintenance or supervisory personnel.

Reporting: The recording of all maintenance work and inspections provide valuable data on the facility condition. A quarterly review of this information will also help to establish more efficient and beneficial maintenance procedures and practices. Along with the written reports, a chain of command for reporting and solving maintenance problems and addressing maintenance needs should be established. From field personnel to the maintenance director, everyone should be encourage to report any problems or suggest any changes to the maintenance program.

6.4.2 CORRECTIVE MAINTENANCE PROCEDURES

Corrective Maintenance is required on an emergency or non-routine basis to correct problems or malfunctions and to restore the intended operation and safe condition of a SWMF.

Removal of Debris and Sediment: Sediment, debris and trash which threatens the discharge capacity of a SWMF should be removed immediately and properly disposed of in a timely manner. Equipment and personnel must be available to perform the removal work on short notice. The lack of an available disposal site should not delay the removal of trash, debris, and sediment. Temporary disposal sites should be utilized if necessary.

Structural Repairs: Structural damage to outlet and inlet structures, trash racks, and headwalls from vandalism, flood events, or other causes must be repaired promptly. Equipment, materials and personnel must be available to perform these repairs on short notice. The immediacy of the repairs will depend upon the nature of the damage and its effects on the safety and operation of the facility. The analysis of structural damage and the design and performance of structural repairs should only be undertaken by qualified personnel.

Dam, Embankment, and Slope Repairs: Damage to dams, embankments, and side slopes must be repaired promptly. This damage can be the result of rain or flood events, vandalism, animals, vehicles, or neglect. Typical problems include settlement, scouring, cracking, sloughing, seepage, and rutting. Equipment, materials and personnel must be available to perform these repairs on short notice. The immediacy of the repairs will depend upon the nature of the damage and its effects on the safety and operation of the facility. The analysis of damage and the design and performance of geotechnical repairs should only be undertaken by qualified personnel.

Dewatering: It may be necessary to remove ponded water from within a malfunctioning SWMF. This ponding may be the result of a blocked principal outlet (detention facility), inoperable low level outlet (retention facility), loss of infiltration capacity, or poor bottom drainage. Portable pumps may be necessary to remove the ponded water temporarily until a permanent solution can be implemented.

Pond Maintenance: Problems such as algae growth, excessive siltation, and mosquito breeding, should be addressed and corrected in a timely manner. The sooner the problem is corrected, the easier it will be to restore a balanced environment in the pond. Due to the complex environment in a pond, it is recommended that agencies such as the U.S. Fish and Wildlife Service be consulted for corrective maintenance procedures.

Extermination of Mosquitoes: If neglected, a SWMF can readily become an ideal mosquito breeding area. Extermination of mosquitoes will usually require the services of an expert, such as the appropriate local city or county department. Proper procedures carried out by trained personnel can control the mosquitoes with a minimum of damage or disturbance to the environment. If mosquito control in a facility becomes necessary, the preventative maintenance program should also be re-evaluated, and more emphasis placed on control of mosquito breeding habitats.

Erosion Repair: Vegetative cover or other protective measures are necessary to prevent the loss of soil from the erosive forces of wind and water. Where a re-seeding program has not been effective in maintaining a non-erosive vegetative cover, or other factors have exposed soils to erosion, corrective steps should be initiated to prevent further loss of soil and any subsequent danger to the stability of the facility. Soil loss can be controlled by a variety of materials and methods, including riprap, gabion lining, sod, seeding, concrete lining and re-grading. The local Soil and Water Conservation District can provide valuable assistance in recommending materials and methodologies to control erosion.

Fence Repair: Fences are damaged by many factors, including vandalism and storm events. Timely repair will maintain the security of the site, however, use of fences should be minimized.

Elimination of Trees, Brush, Roots and Animal Burrows: The stability of dams, embankments, and side slopes can be impaired by large roots and animal burrows. Additionally, burrows can prevent a safety hazard for maintenance personnel. Trees and brush with extensive, woody root systems should be completely removed from dams and embankments to prevent their destabilization and the creation of seepage routs. Roots should also be completely removed to prevent their decomposition within the dam or embankment. Root voids and burrows should be plugged by filling with material similar to the existing materials, and capped just below grade with stone, concrete or other material. If plugging of the burrows does not discourage the animals from returning, further measures should be taken to either remove the animal population or to make critical areas of the facility unattractive to them.

Snow and Ice Removal: Accumulations of snow and ice can threaten the functioning of a SWMF, particularly at inlets, outlets, and emergency spillways. Providing the equipment, materials and personnel to monitor and remove snow and ice from these critical areas is necessary to assure the continued functioning of the facility during the winter months.

6.4.3 AESTHETIC MAINTENANCE PROCEDURES

Aesthetic Maintenance, although not required to keep a SWMF operational, will maintain the visual appeal of a facility and will benefit everyone within the local community. This is particularly true for those SWMFs that are also used by members of the community for athletic and recreational purposes. Aesthetic Maintenance can also reduce the amount of required Preventative and Corrective Maintenance. A comparison of Aesthetic and Preventative Maintenance procedures reveals how both can readily be combined into an overall SWMF maintenance program.

Graffiti Removal: The timely removal of this obvious eyesore will restore the aesthetic quality of a SWMF. Removal can be accomplished by painting or otherwise covering it, or removing it with scrapers, solvents or cleaners. Timely removal is important to discourage further graffiti and other acts of vandalism.

Grass Trimming: Although time consuming, trimming of grass edges around structures and fences will provide for a neat and attractive appearance of the facility. Grass trimming should be scheduled to coincide with grass cutting.

Control of Weeds: Although a regular grass maintenance program will keep weed intrusion to a minimum, some weeds will invariably appear. Periodic weeding will not only help to maintain a healthy turf, but will also keep grassed areas looking attractive. The application of chemical weed control needs to be carefully considered and monitored.

Details: Careful, meticulous, and frequent attention to the performance of maintenance items such as painting, tree pruning, leaf collection, debris removal, and grass cutting will result in a SWMF that remains both functional and attractive.

TABLE 6-3

MAINTENANCE ITEMS BY CATEGORIES

A. PREVENTATIVE MAINTENANCE

1. Grass Cutting
2. Grass Maintenance
3. Vegetative Cover
4. Removal and Disposal of Trash and Debris
5. Sediment Removal and Disposal
6. Mechanical Components
7. Elimination of Mosquito Breeding Habitats
8. Pond Maintenance
9. Inspection
10. Reporting

B. CORRECTIVE MAINTENANCE

1. Removal of Debris and Sediment
2. Structural Repairs
3. Dam, Embankment, and Slop Repairs
4. Dewatering
5. Pond Maintenance
6. Extermination of Mosquitoes
7. Erosion Repair
8. Fence Repair
9. Elimination of Trees, Brush, Roots, and Animal Burrows
10. Snow and Ice Removal

C. AESTHETIC MAINTENANCE

1. Graffiti Removal
2. Grass Trimming
3. Control of Weeds
4. Details

6.5 EQUIPMENT REQUIREMENTS

Table 6-4 lists the Equipment and materials which are typically required to maintain a SWMF. It is presented as a general guide to assist owners, maintenance directors, designers, and financial planners in establishing specific facility maintenance programs. Actual equipment and materials requirements must be determined on an individual basis for each facility. Equipment that is used infrequently could be rented, come from a city wide pool,or be shared with other localities.

Factors to consider in the selection of equipment:

1. Frequency of Usage - Renting or contracting should be considered if the equipment is rarely used.
2. Ease of Operation - The average laborer should able to use the equipment safely without significant training.
3. Economy of Operation - Consider the fuel consumption per hour and the rate of production that the machine offers (i.e. - a riding mower uses x - # of gallons per hour and travels at x - # of feet per minute)
4. Attachments - A machine which can perform a variety of functions can be very cost-effective.
5. Warranty, Service, Parts Availability - Compare warranty periods. Also, equipment which can be serviced by only one local dealer may not be attractive. Finally, ensure that either the dealer stocks a variety of parts for the product line in question or that parts are commonly available on the local market. If the dealer stocks parts for the entire product line, he is more likely to stock a wide range of parts for each product in that line. Commonly available parts reduce the need to rely on one dealer. Down time can slow down or stop maintenance.
6. Transportation - Equipment which can be moved by one employee is ideal. Equipment which requires special trailers or other transporting devices should be avoided if possible. The proper selection of equipment can increase the production and efficiency of maintenance personnel. This can mean a significant reduction in overall maintenance costs.

TABLE 6-4
EQUIPMENT AND MATERIALS COMMON TO MAINTENANCE

A. GRASS MAINTENANCE EQUIPMENT

1. Tractor-Mounted Mowers
2. Riding Mowers
3. Hand Mowers
4. Gas Powered Trimmers
5. Gas Powered Edgers
6. Seed Spreaders
7. Fertilizer Spreaders
8. De-Thatching Equipment
9. Pesticide and Herbicide Application Equipment
10. Grass Clipping and Leaf Collection Equipment

B. VEGETATIVE COVER MAINTENANCE EQUIPMENT

1. Saws
2. Pruning Shears
3. Hedge Trimmers
4. Wood Chippers

C. TRANSPORTATION EQUIPMENT

1. Trucks for Transportation of Materials, equipment, and personnel
2. Trailers for Transportation of equipment

D. DEBRIS, TRASH, AND SEDIMENT REMOVAL EQUIPMENT

1. Loader
2. Backhoe
3. Grader

E. MISCELLANEOUS EQUIPMENT

1. Shovels
2. Rakes
3. Picks
4. Wheel Barrows
5. Fence Repair Tools
6. Painting Equipment
7. Gloves
8. Standard Mechanics Tools
9. Tools for Maintenance of Equipment
10. Office Space
11. Office Equipment
12. Telephone
13. Safety Equipment
14. Tools for Concrete Work (Mixers, Form Materials, etc.)
15. Welding Equipment (for Repair of Trash Racks, etc.)

F. MATERIALS

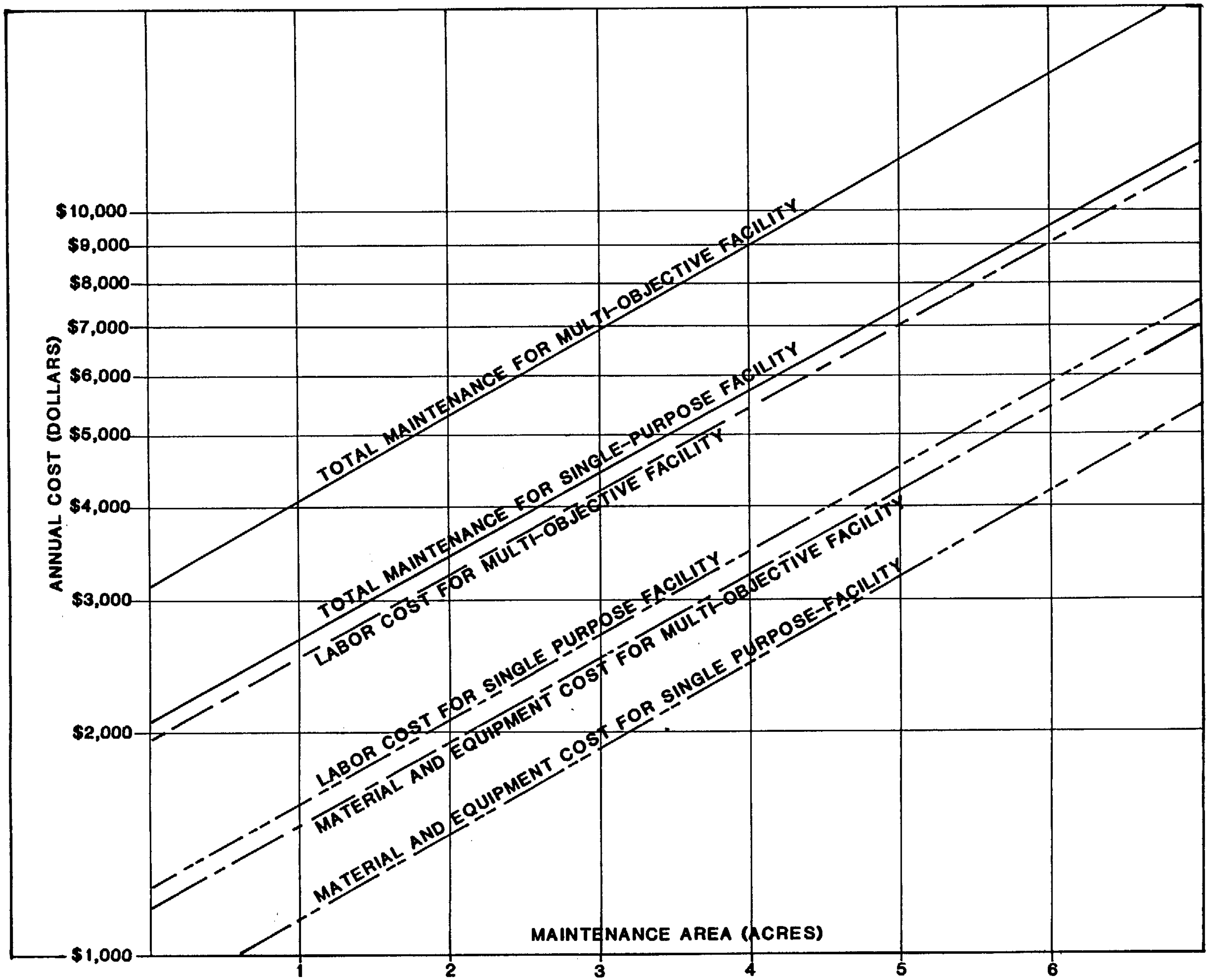
1. Topsoil
2. Fill
3. Seed
4. Soil Amenities (Fertilizer, Lime)
5. Chemicals (Pesticides, Herbicides,)
6. Mulch
7. Paint
8. Paint Removers for Graffiti
9. Spare Parts for Equipment
10. Lubrication
11. Concrete

6.6 MAINTENANCE COSTS

The Figures 6-1 and 6-2 are provided for budgetary purposes. The projected costs were determined for facilities which receive a comprehensive level of maintenance which may be more ideal or thorough than typical. Estimating a relatively high initial budget gives the maintenance director the necessary time and funding to properly establish his maintenance program. As actual needs are experienced, the budget can be adjusted. The date for Figures 6-1 and 6-2 were estimated from data generated from performance standard studies done by URS. hours by various maintenance tasks were multiplied by salary rates including fringe benefits. No allowance for overhead was made. Equipment costs were based on hourly rates typical of rental costs. The maintenance items covered are those listed in Table 6-3.

6.7 TRAINING

Training of the maintenance personnel is very important since they are normally the most frequent visitors to the site. A training program should include: maintenance and inspection techniques; proper record keeping, and stormwater requirements. Particular attention should be paid to the purpose and operation of stormwater management facilities, the importance of thorough maintenance, and the health, safety and other consequences of maintenance neglect.



HAMPTON ROADS PLANNING DISTRICT COMMISSION

FIGURE 6-1
MAINTENANCE COST VS. FACILITY AREA FOR DETENTION BASINS

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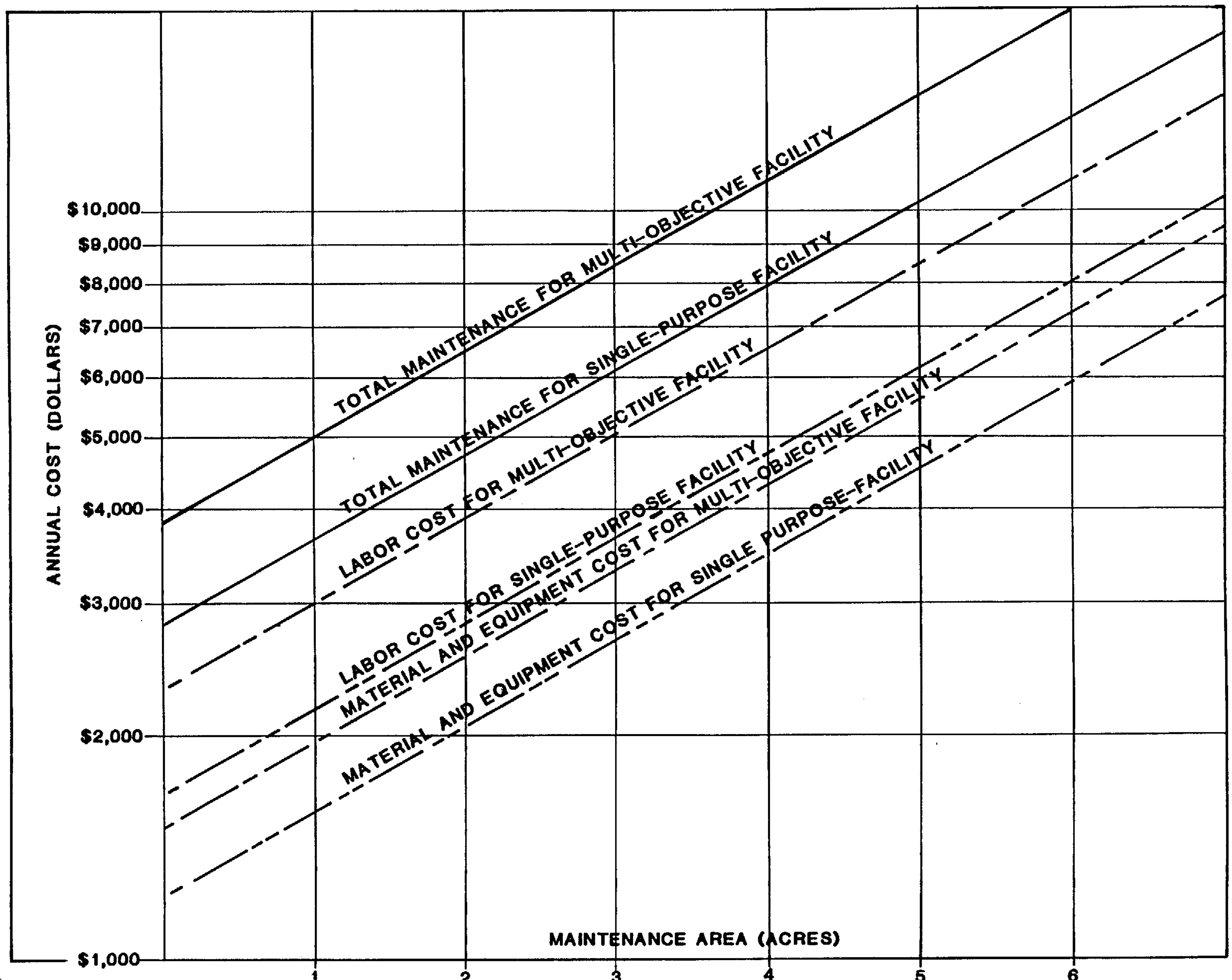

HAMPTON ROADS PLANNING DISTRICT COMMISSION

FIGURE 6-2
MAINTENANCE COST VS. FACILITY AREA FOR RETENTION BASINS

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APPENDIX A

CHECKLIST FOR

STORMWATER MANAGEMENT FACILITIES

PROJECT STATEMENT

- Brief description of the overall project
- Sequence of Construction: Date project is to start, expected dates of soil stabilization, expected date of completion
- Brief description of erosion and sediment control program
- Brief description of stormwater control program

SITE CONDITIONS - PRE-DEVELOPMENT

- Nature and extent of existing vegetation
- Description of soils on site:
    - Include name, texture, slope, depth, drainage and surface area of each type of soil.
- Brief description of sensitive environmental areas located within or in proximity to the site. Such areas include, but are not limited to: resource protection areas and resource management areas, including floodplains, streams, lakes, ponds, wetlands, weak soils, steep slopes, etc.
- Impact analysis to briefly to discuss the ramifications of development.
- See the HRPDC report "Environmental Assessment Procedures", dated 1991 for guidance.

STORMWATER MANAGEMENT PLAN

Description of Plan:

- Brief analysis of problems posed by stormwater runoff on downstream areas
- Pre- and post-development nonpoint source (NPS) loading conditions for CBPA areas and other areas as required by locality

- Selected Best Management Practices (BMPs) or other control procedures and how they were determined; efficiency of such practices; also:
    - Note if use to affect RPA or RMA
    - Procedures for implementing non-structural stormwater BMPs

Hydrologic Calculations:

- Map with existing and proposed drainage areas: Note overland flows over 200' used for computing time of concentration.
- Rainfall data: Include a copy of the Intensity Duration Frequency chart used and provide a list of the intensities used for the selected duration and frequencies. If the Type II SCS rainfall distribution was used, so state.
- Surface runoff coefficients or runoff curve numbers
- Runoff Computation: Note if SCS, rational or other method is used. If some other method is used, supply supporting documentation for that method. Note the times of concentration and how they were determined. Supply calculations sufficient for independent review.
- Base flow for facilities with a permanent pool show calculation for base flow to maintain the required volume for the objective.
- Infiltration: for all storage facilities, show computations for infiltration losses with respect to time.
- Hydrographs and peak flow data: plotted hydrographs from the runoff calculations showing pre-development hydrographs and post-development hydrographs for 2-, 5- and 100-year storms.

Hydraulic Calculations:

- Retention - Detention:
    - Storage volume curves
    - Hydraulic calculations for spillways and outlets
    - For regional or system networks show routing procedure for evaluating impact of discharges on downstream facilities.

- Pipe or culvert structures:
    - Inlet and outlet elevations, slopes
    - Length
    - Diameter or height
    - Mannings roughness coefficient
    - Verification of inlet/outlet control conditions
    - Design flows
- Streams or channels:
    - Map of area and location of cross sections not to exceed 1000' apart with 500' distances preferable. The accuracy of the water surface profiles is greatly dependent upon the selection of cross-sections. The cross sections need to reflect hydraulically controlling cross sections, show overbanks and floodplain limits and project obstacles in the floodplain that may influence flow within the reach between cross-sections.
    - Profile showing stream bottom, top of bank, 2 and 10 year water surface profile; and other profiles as required by locality, e.g., 1000-year floodplain
    - Mannings Roughness Coefficient
    - Velocity
    - Method of computing water surface profile

Structural data for retention - detention facilities:

- Location and design of planned stormwater facilities including verification of structural soundness by a Professional Engineer
- Cross-sections of structures involving embankments; design elevations including freeboard allowances; the composition of core material; include cross-sections of outlet structures; vegetative cover or enclosure
- Cross-sections of infiltration facilities; composition of materials and any type of vegetative cover

- Soil boring data which supports the viability of such facilities
- Statement of applicability of Virginia Dam Safety Act

EROSION AND SEDIMENT CONTROL PLAN

- Vegetative cover to be disturbed
- Estimate of soil loss by use of Universal Soil Loss Equation or other acceptable method. If a different method is used, supply supporting documentation.
- The volume of first flush from the project area and the upstream watershed
- Peak runoff from 10- and 100-year frequency storms based on present and future conditions and according to the existing hazards and degrees of protection required. For watersheds under one square mile, the peak runoff for the 100-year storm is not necessary.
- Methods of calculation
- Phasing of land-disturbing activities:
    - Sequence of land clearing operations
    - Removal and stockpiling of topsoil
    - Major earth moving and grading
    - Control facility installation
- Temporary erosion and sediment control
    - Types of measures and facilities and the rationale for using them
    - Location of each measure or facility with a description of upstream and downstream areas affected
    - Cross-sections or other self-explanatory drawings
    - Calculations supporting these measures
- Permanent erosion and sediment control
    - Types of measures and facilities and the rationale for using them
    - Location of each measure or facility with a description of upstream and

downstream areas affected

- Cross-sections or other self-explanatory drawings
- Calculations supporting these measures

CHESAPEAKE BAY PRESERVATION ACT

- Type of Development (IDA, New, Redev.)
- All components of the RPA (including wetlands and buffers) and the extent of the RMA.
- If compliance is not necessary, supply statement and documentation supporting exemption.
- CBPA Guidance Calculations or locally specified calculations.
- Water Quality Impact Assessment as required.
- Other locality specific documentation.

APPENDIX B

GUIDANCE CALCULATION PROCEDURE

SOURCE: CHESAPEAKE BAY LOCAL ASSISTANCE DEPARTMENT.

INTRODUCTION

This procedure is designed to help applicants determine compliance with a locality's Chesapeake Bay Preservation Act program. This procedure does not supplant any information or requirement of other stormwater management programs, namely any local initiative adopted pursuant to either the Erosion and Sediment Control (ESC) Law [§ 10.1-560, et. seq.] or the Stormwater Management (SWM) Law [§ 10.1-603.1, et. seq.]. While all three programs are intended to protect water resources from further degradation, each requires separate engineering analysis. In general, these programs require calculations as follows:

- a CBPA program : stormwater quality
- a SWM program : stormwater quantity and quality
- an ESC program : two-year design storm runoff volumes and velocities

Many localities may combine all aspects into one, comprehensive program. This calculation procedure would then be just one aspect of that program and a development proposal's submittal.

STEP ONE: **Determine if the site is in a Chesapeake Bay Preservation Area.**

The Regulations[1] require localities to designate Chesapeake Bay Preservation Areas (CBPAs). Guidelines for local designation are contained in Chapters II and III of the *Local Assistance Manual* and Part III of the Regulations. CBPAs consist of two different classifications: Resource Protection Areas (RPAs) and Resource Management Areas (RMAs). The stormwater management criteria apply <u>equally</u> to both RPAs and RMAs.

While localities have flexibility to determine their own CBPAs, those areas will generally include the following land features:

In RPAs: tidal wetlands, nontidal wetlands contiguous to tidal wetlands, tidal shores, tributary streams, a buffer area (of not less than 100 feet), and other lands as designated by the locality;

In RMAs: floodplains, highly erodible soils, highly permeable soils, nontidal wetlands not in the RPA, and other land as designated by the locality.

Determine from the locality's designation maps and criteria if the site is subject to this procedure. Localities may require the entire site to comply with the Regulations even if only a portion of the site is in a CBPA. Determine the locality's requirement on total site compliance.

STEP TWO: **Determine if the site is classified as new development or redevelopment.**

The Regulations provide the following definitions:

Development means the construction, or substantial alteration of residential, commercial, industrial, institutional, recreational, transportation, or utility facilities or structures.

Redevelopment means the process of developing land that is or has been previously developed.

Check with the locality to see if further clarification is provided concerning redevelopment.

NOTE: **Any site in an Intensely Developed Area is automatically classified as redevelopment, regardless of the site's present or previous condition.**
[§ 3.4 of the Regulations]

For development, the post-development nonpoint source pollution runoff load cannot exceed the pre-development load based on "average land cover conditions." This standard can be referred to as a "no net increase" standard. STEP THREE will further discuss "average land cover conditions."

For redevelopment sites not served by BMPs, the post-development non-point source pollution runoff load must be 90 percent or less of the pre-development load for that site. This standard can be referred to as a "10 percent reduction" standard. Redevelopment criteria are not based on average land cover conditions.

For redevelopment sites with BMPs, the following provision(s) must be satisfied to constitute "being served by water quality best management practices":

(1) **In general, runoff pollution loads must have been calculated and the BMP selected for the expressed purpose of controlling NPS pollution.** However, if existing facilities can be shown to achieve the current standard of NPS pollution control, local authorities may consider the site as being served by water quality BMPs.

(2) **If BMPs are structural, facilities must currently be in good working order, performing at the design levels of service.** The local authority may require a review of both the original structural design and maintenance plans to verify this provision. A new maintenance agreement may be required to ensure consistency with the locality's SWM requirements.

STEP THREE: **Determine the relative pre-development pollutant load of the Keystone Pollutant (L_{pre}).**

The Keystone Pollutant for Tidewater Virginia is total phosphorous. The selection of total phosphorous as the keystone pollutant is discussed in Attachment A. For the remainder of this procedure, "pollutant" or "pollutant loading(s)" will mean total phosphorous.

Following development or redevelopment, impervious cover is the key determinant in the levels of pollutant export. Up to 90 percent of the atmospheric pollutants deposited on impervious surfaces are delivered to receiving waters.[2] So, for STEPS THREE and FOUR, the site designer need only determine the amount of total area subject to these criteria and the proposed amount of impervious cover (or equivalent). Guidance on determining equivalents is given in Attachment B. Worksheets A and B will help with these next two steps.

The zoning classification or proposed density of a site will allow applicants to estimate impervious cover. Compliance and final engineering calculations, however, should be based on impervious cover shown on the final site plan. Even so, localities and applicants are encouraged to "err" conservatively, as properties tend to become more impervious with time, e.g. the expansion of a structure, paving a driveway, adding more parking spaces. A conservative estimate indicates more, rather than less, impervious cover. Localities may wish to set a minimum for particular land uses but require the determination of proposed impervious cover and **use the higher number**. Representative land use categories and associated pollutant exports are shown in Table 1.

FOR DEVELOPMENT:

Average Land Cover Conditions ($I_{watershed}$)

Just as a locality must designate CBPAs, a locality must also establish baseloads for watersheds within its jurisdiction. Once set, the baseload will not change unless technology provides a more precise answer. Watershed delineations serve as the baseline for a calculation procedure and do not constitute an additional regulatory step. The two options available to localities are:

GUIDANCE CALCULATION PROCEDURE

1. A locality will designate watersheds within its jurisdiction and calculate the average total phosphorus loading and equivalent impervious cover for each individual watershed, or

2. A locality will declare its entire jurisdiction as part of Virginia's Chesapeake Bay watershed with an average total phosphorus loading (F_{VA}) of **0.45 pounds/acre/year** and an equivalent impervious cover (I_{VA}) of 16 percent.

Some localities may begin with OPTION Two while they gather the necessary data for OPTION ONE. Guidance on how a locality should calculate individual watershed loads is provided in Attachment B. Discussion of the default loadings is in Attachment C.

With $I_{watershed}$, L_{pre} can be calculated using the Simple Method.[3] The derivation of the Simple Method can be found in Appendix A of *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs,* published by the Metropolitan Washington Council of Governments.

$$\boxed{L_{pre} = P \times P_j \times [0.05 + 0.009(I_{watershed})] \times C \times A \times 2.72 / 12}$$

where:

L_{pre} = relative pre-development total phosphorus load (in lbs/yr)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storm with no runoff = 0.9
$I_{watershed}$ = equivalent impervious cover for watershed, or "average land cover conditions" (percent expressed in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)
= 0.26 mg/l when $I_{watershed} < 20$
= 1.06 mg/l when $I_{watershed} \geq 20$
A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion factors

FOR REDEVELOPMENT:

Pre-development loads for redevelopment sites are not based on average land cover conditions. Instead, pre-development loads are based on the site conditions at the time of plan submittal. Therefore, determine existing impervious cover or equivalent.

With $I_{site(pre)}$, L_{pre} can be calculated using the Simple Method.

$$\boxed{L_{pre} = P \times P_j \times [0.05 + 0.009(I_{site(pre)})] \times C \times A \times 2.72 / 12}$$

where:

L_{pre} = relative pre-development total phosphorus load (in lbs)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storm with no runoff = 0.9
$I_{site(pre)}$ = equivalent pre-development impervious cover of the site (percent expressed in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)
= 0.26 mg/l when $I_{site(pre)} < 20$
= 1.06 mg/l when $I_{site(pre)} \geq 20$
A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion numbers

STEP FOUR: **Determine the relative post-development pollutant load (L_{post}).**

Just as with STEP THREE, the designer needs to know the post-development impervious cover (or equivalent). For both new development and redevelopment, post-development loadings are site-specific.

FOR NEW DEVELOPMENT

Again, the Simple Method is used.

$$\boxed{L_{post} = P \times P_j \times [0.05 + 0.009(I_{site(post)})] \times C \times A \times 2.72 / 12}$$

where:

L_{post} = relative post-development total phosphorus load (in lbs)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storms with no runoff = 0.9

$I_{site(post)}$ = equivalent post-development impervious cover (percent in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)
- For Option One: Locally Designated Watersheds
  = 0.26 mg/l when $I_{site(post)} < 20$
  = 1.06 mg/l when $I_{site(post)} \geq 20$
- For Option Two: Va. Chesapeake Bay Default
  =0.26 mg/l for all $I_{site(post)}$

A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion factors

FOR REDEVELOPMENT:

Again, the Simple Method is used.

$$L_{post} = P \times P_j \times [0.05 + 0.009(I_{site(post)})] \times C \times A \times 2.72 / 12$$

where:

L_{post} = relative post-development total phosphorus load (in lbs)
P = average annual rainfall depth (in inches)
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area
P_j = unitless correction factor for storms with no runoff = 0.9
$I_{site(post)}$ = equivalent post-development impervious cover (percent in whole numbers)
C = flow-weighted mean pollutant concentration (in mg/l)
= 0.26 mg/l when $I_{site(post)} < 20$
= 1.06 mg/l when $I_{site(post)} \geq 20$
A = applicable area of site (in ac)

NOTE: 12 and 2.72 are conversion factors

STEP FIVE: **Determine the relative removal requirements (RR).**

Remember from **STEP TWO**, the performance standards are different.

FOR DEVELOPMENT:

$$RR = L_{post} - L_{pre}$$

FOR REDEVELOPMENT:

$$RR = L_{post} - 0.9(L_{pre})$$

If the calculated number is less than or equal to zero, STOP. ***Note that in watersheds using the Tidewater weighted average, F_{VA} = 0.45 lbs/ac/yr, new single-family home parcels one acre or greater do not require BMPs.***

If no BMPS are required, the applicant need only submit documentation to support his or her findings. If such findings are found correct by local officials, the applicant has then satisfied the stormwater management criteria. The state Stormwater Management Law and the Erosion and Sediment Control Law also deal with other water resource related provisions, such as quantity-related requirements.

If removal efficiencies are required, continue on with STEP SIX.

STEP SIX: **Identify BMP options for the site.**

Best Management Practices (BMPs) can be used to remove pollutants. BMPs are not always structural. For instance, trash removal can drastically reduce the amount of solid wastes that reach our streams. However, for the purpose of this discussion BMPs will mean any structural or mechanical device capable of preventing or reducing the amount of pollution from nonpoint sources.

The use of certain BMPs may be limited on some sites by soils, topography, area and other physical characteristics. Most BMPs can only be applied under restricted site conditions. Improperly sited, a BMP cannot perform as designed and may become a chronic maintenance problem. A poorly maintained BMP may even contribute pollutants, e.g. an eroding pond embankment sends sediment into the receiving stream.

BMPs and their associated pollutant removal efficiencies are shown in Table 2. This list is by no means a complete listing of available BMPs, nor does appearance on this list indicate appropriateness for a given situation.

STEP SEVEN: **Determine if feasible BMP options can meet the pollutant removal requirement.**

If runoff from the entire site passes through the BMP, the applicant need only select a BMP with an efficiency rating equal to or greater than the efficiency required [as determined in **STEP FIVE**]. If, as is usually the case, only portions of the site are covered by BMPs, a weighted summation must be made.

Localities may allow pollutant reduction credits for serving off-site areas which drain through BMPs on the subject site. However, while applicants might claim pollutant reduction credits for serving off-site areas, applicants MAY NOT claim credit for one or more off-site BMPs serving their property (even if, in fact, they do). Neither the Act nor the Regulations allow for such an off-set program.

Worksheet C will help with this step of the procedure.

If no combination of BMPs can meet the required standard, the applicant must consider a different site design. Increasing the proportion of site area covered with vegetation is one of the best ways of lowering the required removal efficiencies. A different site layout may make a more appropriate BMP possible; for example, placing structures on "tight" soils may leave more permeable soil for infiltration areas.

ENDNOTES

[1] Chesapeake Bay Local Assistance Board, Final Regulations: VR 173-02-01 *Chesapeake Bay Preservation Area Designation and Management Regulations.* September 1989.

[2] Thomas R. Schueler, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs* (Washington, D.C.: Metropolitan Washington Council of Government, Department of Environmental Programs, 1987), 1.4.

[3] Ibid, 1.9-1.13.

Annual Storm Phosphorous Export — *Table 1*

For Existing Urban Land Uses (in pounds/acre/year)

LAND USES	IMPERVIOUS COVER (%)	ANNUAL RAINFALL (in) 40	41	42	43	44	45
	0	0.11	0.11	0.11	0.11	0.12	0.12
5.0 acre residential lots	5	0.20	0.21	0.21	0.22	0.22	0.23
2.0 acre residential lots	10	0.30	0.30	0.31	0.32	0.33	0.33
1.0 acre residential lots	15	0.39	0.40	0.41	0.42	0.43	0.44
	16	0.41	0.42	0.43	0.44	0.45	0.46
	17	0.43	0.44	0.45	0.46	0.47	0.48
	18	0.45	0.46	0.47	0.48	0.49	0.51
	19	0.47	0.48	0.49	0.50	0.52	0.53
0.50 acre residential lots	20	2.03	2.08	2.13	2.18	2.23	2.28
0.33 acre residential lots	25	2.42	2.48	2.54	2.61	2.67	2.72
0.25 acre residential lots	30	2.82	2.89	2.96	3.03	3.10	3.17
Townhouses	35	3.22	3.30	3.38	3.46	3.54	3.62
	40	3.61	3.70	3.79	3.88	3.97	4.06
	45	4.01	4.11	4.21	4.31	4.41	4.51
Garden Apartments	50	4.41	4.52	4.63	4.74	4.85	4.96
	55	4.80	4.92	5.04	5.16	5.28	5.40
	60	5.20	5.33	5.46	5.59	5.72	5.85
Light Commercial/Industrial	65	5.60	5.74	5.88	6.02	6.16	6.30
	70	5.99	6.14	6.29	6.44	6.59	6.74
	75	6.39	6.55	6.71	6.87	7.03	7.19
	80	6.79	6.96	7.13	7.29	7.46	7.63
Heavy Commercial/Industrial	85	7.98	8.17	8.37	8.57	8.77	8.97
	90	7.58	7.77	7.96	8.15	8.34	8.53
	95	7.98	8.17	8.37	8.57	8.77	8.97
Asphalt/Pavement	100	8.37	8.58	8.79	9.00	9.21	9.42

For Non-Urban Land Uses (in pounds/acre/year)

LAND USE	SILT LOAM SOILS	LOAM SOILS	SANDY LOAM SOILS
Conventional Tillage Cropland	3.71	2.42	0.83
Conservation Tillage Cropland	2.32	1.52	0.52
Pasture Land	0.91	0.59	0.20
Forest Land	0.19	0.12	0.04

STRUCTURAL BMPS FOR CHESAPEAKE BAY PRESERVATION AREAS — TABLE 2

	Acceptable BMP	Average Total P Removal Efficiency
A.	Extended Detention	
	(1) Design 2 (6-12):	20%
	(2) Design 3 (24 hours):	30%
	(3) Design 4 (shallow marsh):	50%
B.	Wet Pond	
	(1) Design 5 (0.5 in/imp.ac):	35%
	(2) Design 6 (2.5 V_r):	40-45%
	(3) Design 7 (4.0 V_r):	50%
C.	Infiltration	
	(1) Design 8 (0.5 in/imp. ac):	50%
	(2) Design 9 (1.0 in/imp. ac):	65%
	(3) Design 10 (2-year storm):	70%
D.	Grassed Swale	
	(1) Design 15 (check dams):	10-20%

These designs are taken from Metropolitan Washington Council of Governments, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, ,1987

Effeciency ratings are taken from John P. Hartigan, P.E., *Three Step Process for Evaluating Compliance with BMP Requirements for Chesapeake Bay Preservation Areas*, 1990

WORKSHEET A : NEW DEVELOPMENT *OPTION ONE: LOCALLY DESIGNATED WATERSHEDS*

1 Compile site-specific data and determine site imperviousness (I_{site}).

			POST-DEVELOPMENT
A*		=	________ acres
I_a:**	structures	=	________ acres
	parking lot	=	________ acres
	roadway	=	________ acres
	other	=	________ acres
		=	________ acres
		=	________ acres
	total I_a	=	________ acres
I_{site} = (total I_a/A) × 100		=	________ (percent expressed in whole numbers)

* Although the area subject to regulations may be only the area actually in a CBPA, some localities may require all of the site to comply with criteria.

** I_a represents the actual amount of impervious area.

2 Determine the average land cover conditions ($I_{watershed}$).

Use $I_{watershed}$ as determined by the locality. If $I_{watershed} < 20$, use C_{pre} = 0.26mg/l. If $I_{watershed} \geq 20$, use C_{pre} = 1.08 mg/l.

3 Determine need to continue.

I_{site} = ________ % (from Step 1)
$I_{watershed}$ = ________ % (from Step 2)

If $I_{site} \leq I_{watershed}$, STOP and submit analysis to this point.
If $I_{site} > I_{waterhed}$, CONTINUE.

4 Set constants.

P_j = unitless rainfall correction factor
= 0.9 for all of Tidewater Virginia

C_{post} = flow weighted mean concentration of total phosphorus
= 0.26 mg/l for $I_{site} < 20$
= 1.08 mg/l for $I_{site} \geq 20$.

P = annual rainfall depth in inches
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolian area
= 45 inches for Hampton Roads area

12 and 2.72 are used in the equation as unit conversion factors.

WORKSHEET A : NEW DEVELOPMENT *Option One: Locally Designated Watersheds*

5 **Calculate the pre-development load (L_{pre}).**

L_{pre} = $P \times P_j \times [0.05 + (0.009 \times I_{watershed})] \times C_{pre} \times A \times 2.72 / 12$

= ____ $\times 0.9 \times [0.05 + (0.009 \times$ ____$)] \times$ ____ $\times$ ____ $\times 2.72 / 12$

= ________ pounds per year

6 **Calculate the post-development load (L_{post}).**

L_{post} = $P \times P_j \times [0.05 + (0.009 \times I_{site})] \times C_{post} \times A \times 2.72 / 12$

= ____ $\times 0.9 \times [0.05 + (0.009 \times$ ____$)] \times$ ____ $\times$ ____ $\times 2.72 / 12$

= ________ pounds per year

7 **Calculate the pollutant removal requirement (RR).**

RR = $L_{post} - L_{pre}$

= ________ - ________

= ________ pounds per year

To determine the overall BMP efficiency required (%RR) when selecting BMP options:

%RR = RR / $L_{post} \times 100$

= (____ / ____) $\times$ 100

= ________ %

WORKSHEET A : NEW DEVELOPMENT

OPTION TWO: VA. CHESAPEAKE BAY DEFAULT

1 Compile site-specific data and determine site imperviousness (I_{site}).

			POST-DEVELOPMENT
A*		=	________ acres
I_a:**	structures	=	________ acres
	parking lot	=	________ acres
	roadway	=	________ acres
	other	=	________ acres
		=	________ acres
		=	________ acres
	total I_a	=	________ acres
I_{site} = (total I_a/A) × 100		=	________ (percent expressed in whole numbers)

* Although the area subject to regulations may be only the area actually in a CBPA, some localities may require all of the site to comply with criteria.

** I_a represents the actual amount of impervious area.

2 Determine the average land cover conditions ($I_{watershed}$).

Use $I_{watershed} = I_{VA}$=16 because $F_{average}$ = 0.45 lbs/ac/yr for Virginia's Chesapeake Bay Watershed. Use C_{pre} = 0.26 mg/l.

3 Determine need to continue.

I_{site} = ________ % (from Step 1)
$I_{watershed}$ = 16 % (from Step 2)

If $I_{site} \leq I_{watershed}$, STOP and submit analysis to this point.
If $I_{site} > I_{watershed}$, CONTINUE.

4 Set constants.

P_j = unitless rainfall correction factor
= 0.9 for all of Tidewater Virginia

C = flow weighted mean concentration of total phosphorus
= 0.26 mg/l for all I_{site}

P = annual rainfall depth in inches
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolian area
= 45 inches for Hampton Roads area

12 and 2.72 are used in the equation as unit conversion factors.

WORKSHEET A : NEW DEVELOPMENT

OPTION TWO: VA. CHESAPEAKE BAY DEFAULT

5 **Calculate the pre-development load (L_{pre}).**

L_{pre} $= P \times P_j \times [0.05 + (0.009 \times I_{watershed})] \times C_{pre} \times A \times 2.72 / 12$

$=$ _____ $\times 0.9 \times [0.05 + (0.009 \times$ _____ $)] \times 0.26 \times$ _____ $\times 2.72 / 12$

$=$ _________ pounds per year

6 **Calculate the post-development load (L_{post}).**

L_{post} $= P \times P_j \times [0.05 + (0.009 \times I_{site})] \times C \times A \times 2.72 / 12$

$=$ _____ $\times 0.9 \times [0.05 + (0.009 \times$ _____ $)] \times 0.26 \times$ _____ $\times 2.72 / 12$

$=$ _________ pounds per year

7 **Calculate the pollutant removal requirement (RR).**

RR $= L_{post} - L_{pre}$

$=$ _________ - _________

$=$ _________ pounds per year

To determine the overall BMP efficiency required (%RR) when selecting BMP options:

%RR $= RR / L_{post} \times 100$

$=$ (_____ / _____) $\times 100$

$=$ _________ %

WORKSHEET B : REDEVELOPMENT

1 Compile site-specific data.

		PRE-DEVELOPMENT	POST-DEVELOPMENT
A*		= ________ acres	= ________ acres
I_a:	structures	= ________ acres	= ________ acres
	parking lot	= ________ acres	= ________ acres
	roadway	= ________ acres	= ________ acres
	other	= ________ acres	= ________ acres
		= ________ acres	= ________ acres
		= ________ acres	= ________ acres
	total I_a	= ________ acres	= ________ acres
$I = (\text{total } I_a / A) \times 100$		= ________ percent expressed in whole numbers	= ________ percent expressed in whole numbers
$R_v = 0.05 + (0.009 \times I)$		= ________ unitless	= ________ unitless
C:	$I \geq 20 = 1.08$ mg/l $I < 20 = 0.26$ mg/l	= ________ mg/l	= ________ mg/l

* Although the area subject to regulations may be only the area actually in a CBPA, some localities may require all of the site to comply with criteria.

2 Set constants.

P_j = unitless rainfall correction factor
= 0.9 for all of Tidewater Virginia

P = annual rainfall depth in inches
= 40 inches for Northern Virginia area
= 43 inches for Richmond Metropolitan area
= 45 inches for Hampton Roads area

12 and 2.72 are used in the equation as unit conversion factors.

3 Calculate the pre-development load (L_{pre}).

$L_{pre} = P \times P_j \times R_{v(pre)} \times C_{pre} \times A \times 2.72 / 12$

= ____ × 0.9 × ____ × ____ × ____ × 2.72 / 12

= ________ pounds per year

4 Calculate the post-development load (L_{post}).

$L_{post} = P \times P_j \times R_{v(post)} \times C_{post} \times A \times 2.72 / 12$

= ____ × 0.9 × ____ × ____ × ____ × 2.72 /12

= ________ pounds per year

5 Calculate the pollutant removal requirement (RR).

$RR = L_{post} - (0.9 \times L_{pre})$

= ________ - (0.9 × ________)

= ________ pounds per year

$\%RR = (RR / L_{post}) \times 100$

= (____ / ____) × 100

= ________ %

WORKSHEET C: COMPLIANCE

Select BMP options using screening tools and list them below. Then calculate the load removed for each option. DO NOT LIST BMPs IN SERIES HERE.

1

Selected Option	Removal Efficiency (%/100)	×	Fraction of CBPA Drainage Area Served (expressed in decimal form)	×	L_{post} (lbs/yr)	=	Load Removed (lbs/yr)
________	________		________		________		________
________	________		________		________		________
________	________		________		________		________

2a

Estimate parameters for non-CBPA drainage areas on the project site (if the locality does not require complete compliance for the whole site). If the locality requires compliance for the whole site, omit this step.

A (on site, non-CBPA) = ________ acres
I_a: structures = ________ acres
parking lot = ________ acres
roadway = ________ acres
other = ________ acres
= ________ acres
= ________ acres

total I_a = ________ acres

$I = (\text{total } I_a/A) \times 100$ = ________ %
$R_v = 0.05 + (0.009 \times I)$ = ________

C: $I \geq 20 = 1.08$ mg/l = ________ mg/l
$I < 20 = 0.26$ mg/l

When using VIRGINIA CHESAPEAKE BAY DEFAULT (F_{V_a} = 0.45 lbs/ac/yr), C=0.26 mg/l for all I_{site}.

2b

Calculate post-development load for on-site non-CBPAs.

$L_{post(outside)}$ $= P \times P_j \times R_v \times C \times A \times 2.72 / 12$

= ____ × 0.9 × ____ × ____ × ____ × 2.72 / 12

= ________ pounds per year

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3 **Determine loadings for off-site areas if the locality allows this option.**

$I_{watershed}$ = from locality <u>OR</u> $I_{watershed} = I_{VA} = 16$

If $I_{watershed} < 20$, use $C_{offsite} = 0.26$ mg/l.
If $I_{watershed} \geq 20$, use $C_{offsite} = 1.08$ mg/l.
If $I_{watershed} = I_{VA}$ use $C_{offsite} = 0.26$mg/l.

$L_{offsite} = P \times Pj \times [0.05 + (0.009 \times I_{watershed})] \times C_{offsite} \times A_{offsite} \times 2.72 / 12$

= _____ × 0.9 × [0.05 + (0.009 × _____)] × _____ × _____ × 2.72 / 12

= _________ pounds per year

4 **Total non-CBPA pollutant loading.**

Step 3 + Step 4 = total non-CBPA loading

_________ + _________ = _________ pounds per year

5 **Calculate credits if the locality allows this option.**

Selected Option	Removal Efficiency (%/100)	×	L_{post} (lbs/yr)	=	Load Removed (lbs/yr)
_________	_________		_________		_________
_________	_________		_________		_________
_________	_________		_________		_________

6 **Calculate overall compliance.**

Step 1 + Step 5 = total load removed

_________ + _________ = _________ pounds per year

If total load removed > removal requirement, criteria are satisfied.

_________ > _________

Revised 7/90

ATTACHMENT A

Many different pollutants can be identified in our streams and water bodies. The Regulations merely require the control of "nonpoint source (nps) pollution." The Model Ordinance defines NPS as pollution consisting of constituents such as sediment, nutrients, and organic and toxic substances from diffuse sources. Trying to deal with all the possible pollutants would make any calculation procedure complicated and expensive. To simplify the calculations needed, a "keystone" pollutant can be selected. A keystone pollutant shares the general characteristics of most other pollutants. By removing the keystone pollutant, other important pollutants will be simultaneously removed. Chapter 2 of *A Framework for Evaluating Compliance with the 10% Rule*[1] reviews each of the major pollutants found in urban runoff for their suitability as the keystone pollutant, based on the following three criteria:

1. The pollutant must have a well-defined adverse impact on the Chesapeake Bay.

2. The pollutant should exist in a "composite" form, i.e. in a roughly equal split between particulate and soluble phases.

3. Enough research data must be available to provide a reasonable basis for estimating how keystone pollutant loads change in response to development and to current stormwater control measures.

The only urban pollutants that appear to meet all three criteria for suitability as a keystone pollutant are: total phosphorus, total nitrogen and zinc (Table 3). Of these three, total phosphorus exists in the most equivalent proportions of soluble and particulate forms (40/60). Total nitrogen and zinc are less proportionate, at 20/80 and 25/75, respectively.

TABLE 3

Pollutant	Well-Defined Impacts on the Bay?	Composite Form?	Adequate Data?
Sediment	yes	no	no
Total Phosphorous	yes	yes	yes
Total Nitrogen	yes	yes	yes
Coliform Bacteria	yes	no	no
BOD/COD	yes	yes	no
Oil/Grease	yes	no	no
Zinc	yes	yes	yes
Lead	yes	no	yes
Toxics	no	no	no

By removing total phosphorus, an equal or greater level of removal for most other urban pollutants is simultaneously obtained. An equal or higher level or removal is possible for nearly every other pollutant, except total nitrogen. Total nitrogen is primarily found in soluble form, which is much more difficult to remove with current techniques. Nevertheless, by removing phosphorus, a reasonable degree of nitrogen is still removed as well.

Based on this review, total phosphorus was selected as the best candidate for the keystone pollutant in Tidewater Virginia. In doing so, Virginia will target the same pollutant as Maryland, preserving some consistency in our multi-state Bay preservation effort.

ENDNOTE:

[1] Schueler, Thomas R. and Matthew R. Bley, *A Framework for Evaluating Compliance with the Chesapeake Bay Critical Area* (Washington, D.C.: Maryland Critical Area Commission and Maryland Department of the Environment, 1987).

ATTACHMENT B

The Regulations require new development stormwater management criteria be based on "average land cover conditions." Watershed designations serve as the baseline for a calculation procedure and do not constitute an additional regulatory step. Localities will have two options:

1. A locality will designate watersheds within its jurisdiction and calculate the average phosphorus loading and impervious cover for each individual watershed, or

2. A locality will declare its entire watershed as part of Virginia's Chesapeake Bay watershed with an average phosphorus loading of 0.45 pounds/acre/year and impervious cover of 16 percent.

A locality may begin with Option Two while they gather the necessary data for Option One. Figure 1 shows how Fairfax County could break up its watersheds. This discussion revolves around Option One. Option Two is discussed in Attachment C.

To determine average land cover conditions within a watershed, the locality must follow a three-step procedure:

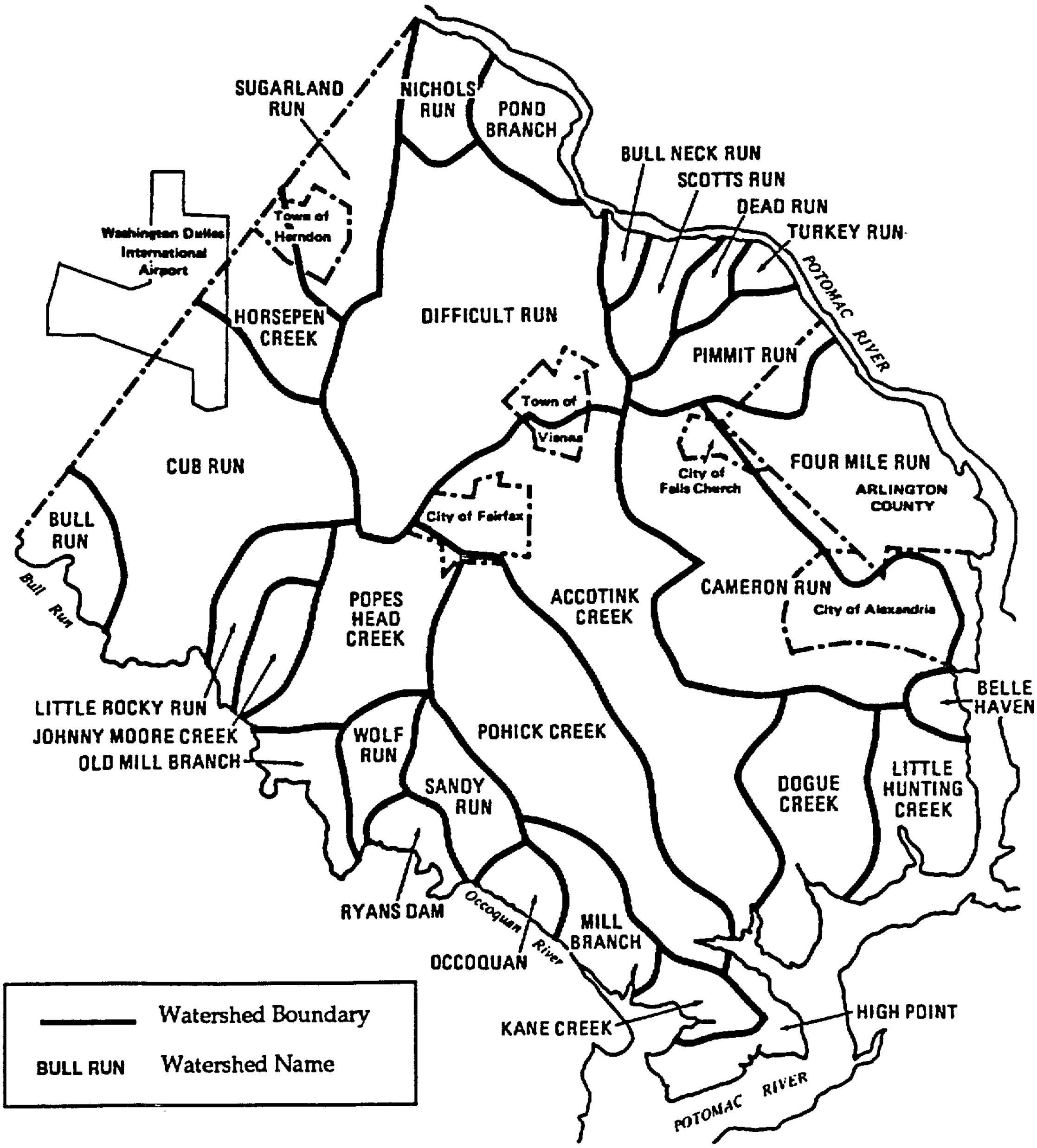
1. **Evaluate individual watersheds.** We recommend a minimum watershed area of 100 acres. Localities may wish however, to use watershed delineations used for other aspects of its work, e.g. a sanitary sewer master plan.

2. **Know existing land use data.** The Regulations are based on present land uses, not proposed land uses. A comprehensive plan is more future oriented than a zoning map. Still, a zoning map does not always indicate present use. A locality may also be able to use current aerial photographs. Data may be cross-referenced with Commissioner of Revenue information.

3. **Compute a weighted average of impervious cover (or its equivalent).** The Simple Method (and the nonpoint source pollution load) is highly dependent on the percent of impervious cover. Some land uses contribute nonpoint source pollution but do not have "impervious covers," e.g. forest and agriculture lands. Therefore, conversions, or equivalents, must be determined. Use Table 1 to find equivalent loading/impervious factors for non-urban uses. Localities may use other documented loading factors, especially if found to be more appropriate to that locality, as long as the factors are used consistently.

   Weighted averages are frequently computed for quantity related analyses and this process is identical. Figure 2 shows how average land cover conditions might be calculated for a 100-acre watershed.

POSSIBLE FAIRFAX COUNTY WATERSHEDS — *FIGURE 1*



Source: County of Fairfax, *1987 Annual Report on the Environment* (Fairfax, Va.: Environmental Quality Advisory Council and Office of Comprehensive Planning, 1987), p. 16

CALCULATING AVERAGE LAND COVER CONDITIONS — FIGURE 2

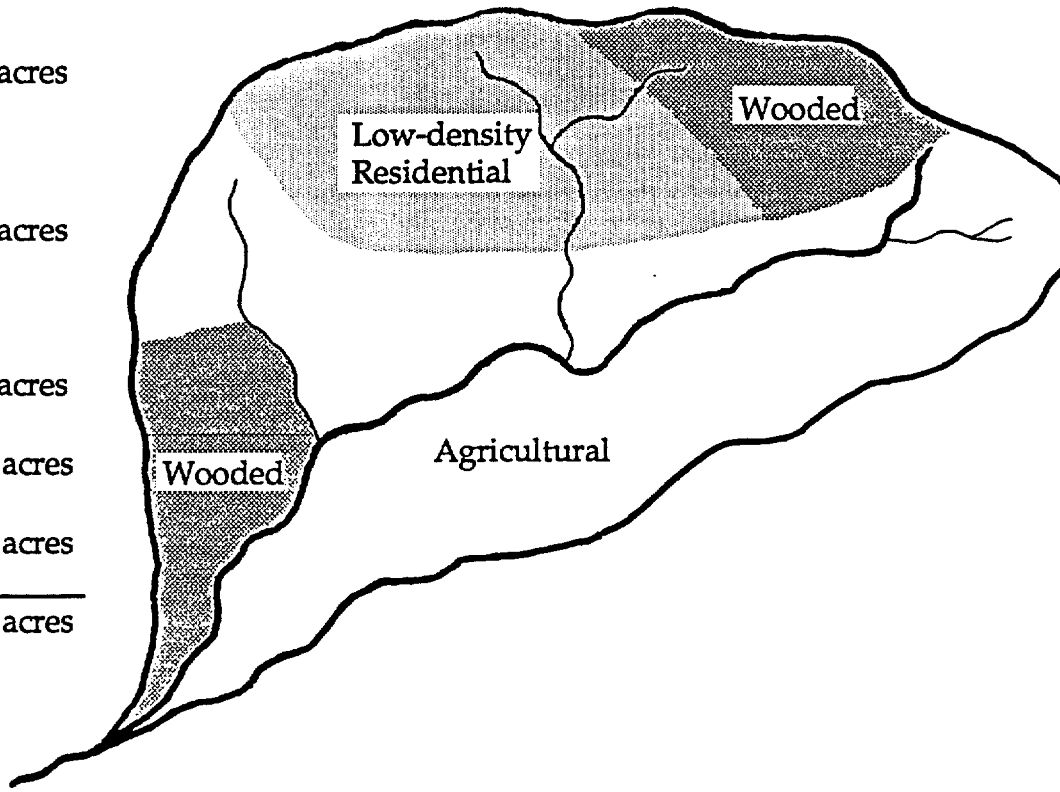
100 acre Watershed

Wooded = 20 acres

Low-density Residential = 20 acres
(1-acre lots)

Agriculture
- Pasture = 30 acres
- Conservation tillage = 15 acres
- Conventional tillage = 15 acres

Total acreage 100 acres



Land Use	Loading: * lbs/acre/year	# of Acres	Weighted Load: lbs/year
Wooded	0.12	20	2.4
1-acre lots	0.42	20	8.4
Pasture	0.59	30	17.7
Conventional	2.42	15	36.3
Conservation	1.52	15	22.8
		100	87.6

* Phosphorous; based on rainfall of P=43 inches/year and loam soils.

$$\Sigma = \frac{\text{Sum of weighted loadings}}{\text{total acreage}}$$

$$= \frac{0.12(20) + 0.42(20) + 0.59(30) + 2.42(15) + 1.52(15)}{20 + 20 + 30 + 15 + 15} = \frac{88 \text{ lbs per year}}{100 \text{ acres}} = 0.88 \text{ lbs per acre per year}$$

$$\text{Equivalent Impervious Cover} = I_{watershed} = 19$$

ATTACHMENT C

Not all localities will have the ability to designate individual watersheds and compute an average watershed baseload. For that reason, the department has determined a default load for Tidewater Virginia.

Following the procedure outlined in Attachment B:

1. Designate watershed.

   The department chose the entire Virginia portion of the Chesapeake Bay watershed – not just Tidewater Virginia (as defined by the Chesapeake Bay Preservation Act). The department encourages multi-jurisdictional cooperation among localities to designate large-scale watersheds as well.

2. Evaluate existing land use data.

   Existing land use data is given in ***Virginia's Chesapeake Bay Initiatives: First Annual Progress Report*** (September 1985) produced by the Virginia Council on the Environment. This breakdown is shown in Figure 3.

3. Compute a weighted average of impervious cover (or its equivalent).

   Because urban areas are most likely to adopt Option One, urban areas are excluded from the weighted average. In addition, loading rates for "urban" areas are highly variable.

F_{VA} = relative total phosphorus load for Virginia's Chesapeake Bay watershed

$F_{(X)}$ = relative total phosphorus load for any land use (X)

$$F_{VA} = \%FOR(F_{FOR}) + \%PAST(F_{PAST}) + \%CST(F_{CST}) + \%CVT(F_{CVT})$$

$$= 0.66(0.12) + 0.21(0.59) + 0.07(1.52) + 0.06(2.42)$$

$$= 0.45 \text{ lbs/ac/yr}$$

Use Table 1 to determine the equivalent impervious cover. The average loading, F_{VA} = 0.45 lbs/ac/yr, falls between impervious covers of 16 to 18 percents. Because of the differing annual rainfall across the state, the department has choosen the most conservative value of 16.

$$F_{VA} = 0.45 \text{ lb/ac/yr} \iff I_{VA} = 16\%$$

Therefore, the default load for Virginia's Chesapeake Bay watershed is 0.45 lb/ac/yr with an equivalent impervious cover of 16 percent. Localities are encouraged, but not required, to customize this aspect of the procedure, even if computing individual watersheds is not feasible. The Town of Herndon might use $I_{VA} = 18$, Caroline County might use $I_{VA} = 17$ and Isle of Wight County would retain $I_{VA} = 16$.

Virginia Land Use Data — *Figure 3*

River Basin	total area (sq.mi.)	% URB	URB area (sq.mi.)	% FOR	FOR area (sq.mi.)	% PAST	PAST area (sq.mi.)	% CST	CST area (sq.mi.)	% CVT	CVT area (sq.mi.)
Potomac	14670	7	1027	56	8215	26	3814	7	1027	4	587
Rappahannock	2630	1	26	64	1684	20	526	8	210	7	184
York	2980	0.2	6	70	2090	13	388	10.1	302	6.7	200
James	10495	3	315	73	7661	14	1469	6	630	4	420
Eastern Shore	1000	1.5	15	50	500	805	85	9	90	31	310
Total (w/urban)	31781	5	1389	63	20150	20	6286	7	2259	5	1701
Total (w/o urban)	30398	n/a	n/a	66	20150	21	6286	7	2259	6	1701

URB = urban land uses
FOR = forest cover
PAST = pasture land
CST = conservation till acreage
CVT = conventional till acreage

Source: Commonwealth of Virginia, Council on the Environment, *Virginia's Chesapeake Bay Initiatives: First Annual Report* (Richmond, Va.: Council on the Environment, 1985).